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Risk-Based Bridge Inspection Practices

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ABSTRACT

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Improving bridge safety, reliability, and the allocation of bridge inspection resources are the goals of the proposed risk-based bridge inspection practices. Currently, most bridges in the United States are inspected at a fixed calendar interval of 24 months, without regard to the condition of the bridge. Newer bridges with little or no damage are inspected with the same frequency as older, more deteriorated bridges thus creating inefficiency in the allocation of inspection resources. Because of limited resources, it is not possible to spend the necessary time examining bridges that are in poor condition and require extra attention since equal effort is also spent on bridges in good condition. In addition, no quantitative evidence exists to suggest that the 24 month inspection interval is the appropriate interval to achieve the desired level of safety.

The proposed methodology incorporates reliability theory and expert elicitation from the Indiana Department of Transportation's Risk Assessment Panel, developed during this research, to rationally determine bridge inspection needs. Assessments are made based on the likelihood and consequence of failure for specific bridge components. The likelihood of failure is determined through attributes based on design, loading, and condition characteristics while the consequence of failure is based on expected structural capacity, public safety, and serviceability. By combining the expressions of likelihood and consequence for each component, an optimum inspection interval for the entire bridge can be determined through the use of risk matrices.

The methodology was evaluated through case studies involving Indiana bridges. Over 30 years of historical inspection reports were utilized in the back-casting process to

evaluate deterioration levels and assess the adequacy of the risk criteria. Results of the case studies conducted during the research indicated that the risk analysis procedures provided suitable inspection intervals ranging from 24 to 72 months for Indiana bridges.

CHAPTER 1. INTRODUCTION

1.1. Background & Organization

Improving bridge safety, reliability, and the allocation of bridge inspection resources are the goals of the proposed risk-based bridge inspection practices. Currently, bridges in the United States are inspected at a fixed calendar interval of 24 months, without regard to the condition of the bridge. Newer bridges with little or no damage are inspected with the same frequency as older, more deteriorated bridges thus creating inefficiency in the allocation of inspection resources and limiting the resources that can be spent on bridges requiring extra attention.

The proposed methodology incorporates risk theory and expert elicitation from the Indiana Department of Transportation's Risk Assessment Panel to rationally determine bridge inspection needs. Assessments are made based on the likelihood and consequence of failure for specific bridge components. The likelihood of failure is determined through attributes based on design, loading, and condition characteristics while the consequence of failure is based on expected structural capacity, public safety, and serviceability. By combining the expressions of likelihood and consequence for each component, a maximum inspection interval for the entire bridge can be determined through the use of risk matrices.

This document is organized into seven chapters plus appendices. Chapter 2 provides an in-depth literature review on reliability and risk-based inspection approaches and background on current bridge inspection processes. Chapter 3 describes key elements of risk-based inspections including occurrence and consequence factors. Chapter 4 summarizes the Indiana Department of Transportation's Risk Assessment

Panel meeting and workshop. Chapter 5 contains the results and interpretation of the back-casting of Indiana bridges. Chapter 6 presents the proposed families of bridges for the INDOT inventory and an implementation strategy. Lastly, Chapter 7 describes the results, conclusions and recommendations from this research study.

Appendix A contains a summary of the attributes and damage modes determined from the INDOT Risk Assessment Panel meeting. Guidelines for determining the consequence factor and consequence factor tables can be found in Appendix B. The back-casting results with detailed bridge information for Indiana bridges can be found in Appendix C.

1.2. Research Objectives

The research objectives for this project are as follows:

- Develop criteria for a risk-based assessment including the development of damage modes and key attributes for the likelihood and consequence factors used in setting inspection intervals.

- Refine the risk assessment through the use of a Risk Assessment Panel (RAP) comprised of INDOT engineers, inspectors, and consultants.

- Verify the developed risk assessment model through back-casting by using historical inspection records and tracking deterioration progress to assess the adequacy of the selected risk criteria.

- Develop criteria for families of bridges to facilitate future risk assessments.

CHAPTER 2. CRITICAL REVIEW OF LITERATURE

The objective of the literature review was to assemble and review research in regard to reliability and risk-based bridge inspection practices. This includes a review of current bridge inspection practices along with previous research completed on various reliability and risk-based approaches. Additionally, risk-based methodologies in other industries were considered.

This literature review begins with a brief perspective on historical and current bridge inspection practices, followed by an overview of inspection practices in Europe. Previous research on risk approaches for the bridge industry, including the NCHRP 12-82 study, is presented.

2.1. Overview of Bridge Inspection Intervals

Current bridge inspection practices were inspired in part by the catastrophic Silver Bridge collapse in Point Pleasant, West Virginia on December 15th, 1967. Prior to the collapse, little focus was given to bridge safety inspections and maintenance. After the collapse, national interest prompted Congress to include a section in the Federal Highway Act of 1968 that created a national bridge inspection program. In 1971, the National Bridge Inspection Standards (NBIS) was created and established a mandatory maximum 24 month inspection interval as well as maintenance recommendations (*National Bridge Inspection Standards,* 2004). The two year interval was based on engineering judgment and experience. Bridge owners are also currently given the option of shorter inspection intervals for older and more deteriorated bridges (*Bridge Inspector's Reference Manual,* 2012).

A few minor changes have been made to inspection practices over the years. Currently, the Federal Highway Administration (FHWA) allows inspection intervals to be extended to 48 months for bridges meeting certain criteria and gaining approval from the agency (FHWA, 1995). Even with these changes, most bridge owners utilize a typical 24 month inspection cycle for the majority of their inventory. Additionally, the NBIS created different bridge inspection categories including Initial, Routine, Damage, In-Depth, Fracture Critical, Underwater, and Special (*Bridge Inspector's Reference Manual,* 2012*)*:

- *Initial*: The first inspection of a bridge after construction or a reconfiguration such as widening or lengthening. This inspection provides baseline conditions and identifies existing problems.

- *Routine*: Regularly scheduled inspections to ensure the safety of the structure and ensure service requirements are met. This inspection typically consists of observations and/or measurements of the functionality of the bridge and notes any deviations from previously recorded conditions.

- *Damage*: An unscheduled inspection to assess structural damage resulting from human actions or environmental factors. The inspection assesses if load restrictions, closure of the bridge, or repairs are necessary.

- *In-Depth:* A close-up inspection to identify deficiencies not found during a routine inspection. This inspection can include non-destructive testing and/or load rating. Inspections can occur as a follow-up to the routine inspection or can be scheduled independently.

- *Fracture Critical*: An arm's length inspection of bridges where failure of an element is expected to result in a partial or complete collapse. This inspection detects cracks using visual or nondestructive testing methods.

- *Underwater*: An inspection of the underwater portions of the bridge that generally require diving. Scour is evaluated as well as structural damage, erosion, ice loading, debris accumulation, and navigation traffic collision.

- *Special:* An inspection scheduled at the discretion of the bridge owner. Typically, special inspections monitor known or suspected deficiencies.

This uniform interval approach has advantages and disadvantages. Most importantly, safety, serviceability, and reliability appear to have been maintained nationwide. In addition, the calendar-based approach allows for ease in scheduling inspections. However, the interval and scope of the inspections do not account for bridge age, design, or environment. Often, an older bridge will display advanced levels of deterioration when compared to a younger bridge. Modern designs utilize improved materials, including increased durability and resistance to fatigue and fracture. Environment also plays a huge role in bridge deterioration, as bridges in aggressive environments with chloride exposure will deteriorate at a quicker rate than bridges in arid environments. Accounting for the variability in design, detailing, and operating conditions would allow for customized inspection requirements that improve bridge safety and reliability, as well as optimize resources for bridge inspection.

2.2. Bridge Inspection Practices in Other Countries

A brief summary of bridge inspection practices in Europe is presented here for reference. Inspection practices in other countries permit bridge inspection intervals up to six years.

2.2.1. Finland

Simple safety inspections are conducted annually. Primary inspections occur every five years, as do underwater inspections, if applicable. More frequent inspections occur for bridges in poor condition or for important bridges (*Bridge Evaluation*, 2008).

2.2.2. Sweden

Major inspections are performed every six years, and are arm's length inspections. A decision about the inspection interval is made after the major inspection, and deteriorating bridges are inspected more frequently. A general inspection is a follow-up inspection for the major inspection, and occurs in the interval between major inspections (*Bridge Evaluation*, 2008).

2.2.3. Germany

Basic inspections are performed by maintenance personnel and occur at frequent but undefined intervals. Visual inspections occur every three years, and major inspections involving material testing and an in-depth visual inspection occur every six years (*Bridge Evaluation*, 2008).

2.3. Proposed Reliability Based Inspection Strategies

As the interest in optimizing bridge maintenance and inspection practices grew, so did the interest in reliability-based inspection methodologies. As a result, a number of methodologies have been proposed to ensure structural safety while also restructuring the inspection process for efficiency. Three different approaches are presented along with a case study.

2.3.1. Risk-Ranking Bridge Inspection Strategy

The main objective of Stewart's study was to assess structural safety using a risk-ranking method.

- The assessment began with inspection and testing of a bridge to determine the random variables, including material types, current condition, and resistance parameters. A target reliability index was selected based upon a calibration procedure and structural costs were considered (Stewart et al., 2001). However, the strategy did not account for serviceability issues that also affect the performance of the bridge.

- On the loading side, live load effects are assumed to be randomly distributed and were modeled by a Poisson point process. On the resistance side, a cumulative-time failure probability was created for corrosion of steel in reinforced concrete beams (Enright & Frangopol, 1998). Using these time-variant models was beneficial for the risk-ranking method (Stewart et al., 2001).

- The major component of the performed risk ranking was the expected cost of failure. This included traffic delay and disruption costs and the costs to the structure. It was recommended that the analysis is updated every 5-10 years to account for deterioration or changing conditions at the bridge. Ultimately, it was concluded that risk ranking methods can be used to select bridges for maintenance, repair, and replacement based upon safety concerns (Stewart et al., 2001).

- Target reliability indices can be difficult to define due to multiple limit states, loading variables, the as-built condition, and probability models. The index only has meaning within the context of the given methodology and must be calibrated. This limits the effectiveness of the index (Stewart et al., 2001).

2.3.2. Time-Dependent Bridge Inspection Strategy

The main objective of Akgul and Frangopol's study was to investigate the time-dependent relationship between load rating factors and reliability indices using rating-reliability profiles and rating-reliability interaction envelopes.

- A time-dependent live-load model was developed using weigh-in-motion studies. It compared favorably to the current bridge design code distribution factors. Previously developed deterioration models for corrosion in concrete and steel bridges were used to determined bridge capacity. A Monte Carlo simulation was used to provide a reliability analysis. Updating the models with current inspection data increased the accuracy and improved decisions for maintenance, repair, and replacement (Akgul & Frangopol, 2004).

- A bridge network in Colorado demonstrated the application of the analysis (Akgul & Frangopol, 2004).

- Individual bridges and the overall system reliability were considered in the formation of the load rating predictions. The reliability envelopes permit engineers to calculate the live-load capacity and reliability index predictions for any bridge in the inventory in a given period of time. To ensure safety, it was recommended that the bridge evaluation is based on the reliability index as opposed to the live-load capacity. The time-dependent approach was considered a useful tool to determine reliability of bridges in Colorado (Akgul & Frangopol, 2004).

2.3.3. Probability-Based Bridge Inspection Strategy

The main objective of Sommer's study was to develop a probability-based procedure for bridge girders, specifically steel girders subject to corrosion.

- Failure modes used for this analysis were bending failure, shear failure, and bearing failure. The reliability of the girder was modeled using a series system with built-in safety margins (Sommer et al., 1993).

- An optimization study was also completed for each girder to find the inspection interval. It included expected costs over the lifetime of the bridge including inspection, maintenance, repair, and replacement costs (Sommer et al., 1993).

- A case study involving a typical steel highway girder bridge was evaluated using the probability-based procedure. The parameters for failure were modulus of elasticity, steel yield stress, compressive strength of concrete, and corrosion functions. Traffic live load was used to define shear and moment demand (Sommer et al., 1993).

- The conclusion was that all girders can be inspected at the same interval and that after twenty years corrosion will be the controlling variable in the analysis. It was suggested that the best inspection interval would be between five and ten years (Sommer et al., 1993).

2.3.4. Case Study: Repair Optimization for a Colorado Highway Bridge

The main objective of Estes and Frangopol's study was to determine a system reliability approach to optimizing the repair strategy for highway bridges.

- First, the relevant failure modes of the bridge were determined and limit-state equations were used to establish the likelihood of each failure mode. In addition, deterioration and live-load models were developed to compute the reliability of the bridge over time. Repair or replacement criteria were also created. An optimized repair schedule was then developed using the models and current inspection data (Estes & Frangopol, 1999).

- Colorado bridge E-17-AH was the bridge used as a case study. Random variables including yield strength of steel, effective depth of rebar in concrete, 28-day compressive strength of concrete, and impact on girders were used to determine the likelihood of each failure mode. Deterioration models considered were a corrosion propagation model for superstructure and a salt penetration model for substructure. The optimum repair strategy suggested was to replace the deck after 50 years, and replace the bridge after 94 years with intermediate repairs when the system reliability fell below the target value (Estes & Frangopol, 1999).

- A limitation of the study was the strength-based and not serviceability-based nature of the analysis. Serviceability concerns often control for repair intervals. Additionally, detailed inspections must be performed to accurately update the model. Finally, more research is required to build confidence in the analysis (Estes & Frangopol, 1999).

2.4. NCHRP 12-82: Risk-Based Bridge Inspections

The primary objective of the NCHRP 12-82 study was to develop a methodology for Risk-Based Inspection (RBI) of highway bridges. Additional goals were to improve the safety and serviceability of bridges and to focus inspection resources where most needed. This was accomplished through a semi-quantitative framework that included a calculation of the likelihood of failure through a points system and the determination of consequence factor through expert judgment (Washer & Connor, 2014).

2.4.1. Process

An expert panel was established at the owner level to assess the reliability characteristics of bridges within the state. The expert panel used engineering rationale to determine typical deterioration patterns and the potential outcomes of damage. This was accomplished in three simple steps (Washer & Connor, 2014):

1. Determine what can go wrong: Possible damage modes are identified for the elements of the bridge. The likelihood of damage falls into one of four categories—Remote, Low, Moderate, or High—based upon design, loading, and condition characteristics.

2. Determine the consequence: Assuming the given damage mode occurs, the consequence is assessed in terms of safety and serviceability. The consequence of damage falls into one of four categories—Low, Moderate, High, or Severe—based upon the immediate and short-term effects to structural safety and public safety.

3. Determine inspection interval and scope: Based upon the results from the determination of likelihood and consequence, a four by four matrix is utilized to determine inspection needs and assign an inspection interval.

2.4.2. Risk Assessment Panel

Expert elicitation plays an important role in determining the likelihood of damage and the level of associated consequences. For complex systems subjected to complex working environments where little data are available, expert elicitation is a consensus based approach to quantify uncertainty. The petroleum and mechanical industries use risk-based methods for their processes (Washer & Connor, 2014). For determining bridge reliability, NCHRP 12-82 proposed the risk assessment panel be conducted at the owner level of the bridge inventories. Suggestions of members to include on the panel were: a bridge inspection expert, a bridge management engineer, a bridge maintenance engineer, a materials engineer, a structural engineer, an independent expert, and a facilitator. Because bridge inventories vary widely between states, and even within a state, it is important that panel members are familiar with the local considerations for damage modes and consequences.

2.4.3. Occurrence Factor

The occurrence factor is the likelihood of failure during the inspection interval. In the NCHRP 12-82 report, the occurrence factor was determined by a simple scoring system based upon the attributes of the bridge (Washer & Connor, 2014). Attributes that were considered less reliable are given points, while more reliable attributes do not score. The Risk Assessment Panel (RAP) determines the point value for each attribute, and which attributes to include in the framework, based upon their experience and judgment. Therefore, higher point values correspond to a higher occurrence factor. The occurrence factor has four categories: Remote, Low, Moderate, and High (Washer & Connor, 2014).

2.4.4. Consequence Factor

The consequence factor is the expected or anticipated outcome if a failure of a bridge element were to occur. In NCHRP 12-82, the consequence factor was determined during the RAP meeting and fell into one of four categories: Low, Moderate, High, or Severe (Washer & Connor, 2014). Low consequence events were expected to have no effect on safety and a minor effect on serviceability. Severe consequence events are catastrophic in nature, and were expected to have a major effect on safety and serviceability. Factors considered when determining consequence factor include structural characteristics, material characteristics, traffic loading, and bridge environment (Washer & Connor, 2014).

2.5. Summary

The primary intent of the literature review was to provide a basic background on reliability and risk-based methodologies and applications to the bridge industry. Inspection practices in other countries were examined as a basis for comparison. Risk-based approaches have been successfully implemented in other industries, and a risk-based approach is a viable option for the bridge industry as well.

As the bridge inventory ages, new strategies will be needed to maintain safety and reliability. Multiple reliability approaches were discussed, including risk ranking, time-dependent, and probability based methods. However, these methods are not easily integrated with the current inspection process. Therefore, a method that encompasses the theories behind risk-based inspection while also having a simple implementation procedure is needed. NCHRP 12-82 attempts to fill that void, and offers a risk-based approach that maintains safety and reliability, optimizes inspection and maintenance resources, and provides an easily integrated solution. This research utilizes NCHRP 12-82 as a framework for the risk methodology and customizes the process to meet Indiana bridge inventory and inspection needs.

CHAPTER 3. KEY ELEMENTS OF RISK-BASED INSPECTIONS

Key elements of risk-based inspections include the risk assessment panel (RAP), the occurrence factor, the consequence factor, and the inspection procedures. The risk assessment panel determines the scope and framework for bridge risk assessments. Occurrence factor and consequence factor are part of the framework and are used to determine the inspection interval. Inspections provide up-to-date information for the risk assessment. Each of these elements is described in this chapter along with a general overview of risk theory.

3.1. Risk Overview

The risk methodology developed through NCHRP 12-82 (Washer & Connor, 2014) and examined herein was designed to ensure bridge safety, optimize the inspection process, be easily implemented, meet the needs of different states, and utilize existing knowledge of in-service bridge behavior. With these considerations, an approach that combined owner insight, probabilistic structural reliability theories, and qualitative risk analysis was developed.

Three primary questions comprise the methodology.

- *What can go wrong and how likely is it?* Based upon the forms of deterioration observable in a bridge and their related design, loading, and condition characteristics, the likelihood of damage can be determined. The likelihood of damage is also known as the occurrence factor and can be classified as one of four categories. These comprise the occurrence factor.

- *What are the consequences?* Assuming the damage occurs, consequence is assessed in terms of safety and serviceability. The immediate and short term consequences are also considered. Consequence is classified as one of four categories. These comprise the consequence factor.

- *What is the inspection interval and scope?* Using a risk matrix, the expressions from occurrence factor and consequence factor are combined to prioritize inspection needs and assign an inspection interval for the bridge.

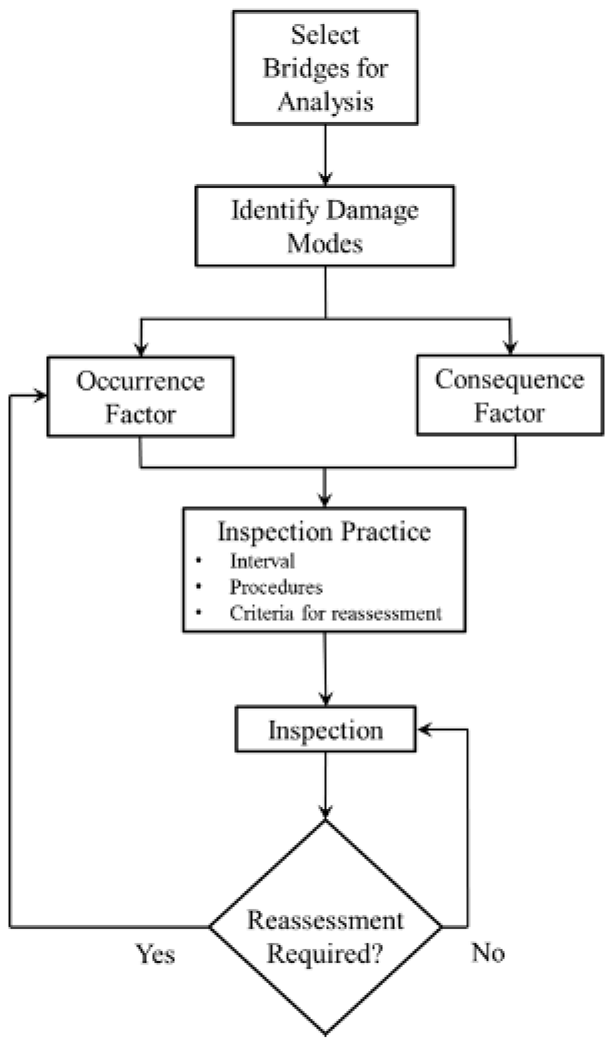

Figure 3.1: Risk-based Inspection Flowchart. Adapted from NCHRP 12-82: *Developing Risk-Based Bridge Inspection Practices*.

The risk-based inspection process is shown in the flowchart in Figure 3.1. After selecting a bridge for analysis, the damage modes are determined for the bridge components based upon engineering experience, design characteristics, material characteristics, and environment using the information obtained from the RAP. To categorize the likelihood of serious damage developing over time, key bridge component characteristics, or attributes, are identified and scored based upon importance. The result is known as the occurrence factor. An assessment of the consequence factor occurs concurrently, but independently. By combining the occurrence factor and consequence factor in a risk matrix similar to Figure 3.2, the inspection interval can be determined. The longest inspection intervals occur when both the occurrence factor and consequence factor are in the green area, or low. Bridges with high occurrence and high consequence factors have inspection intervals in the red area and are inspected the most often. Bridges with a high occurrence factor and low consequence factor or a low occurrence factor and a high consequence factor have an inspection interval between the two extremes in the yellow area. This rational approach focuses inspection efforts on bridges where safety or serviceability are likely to be disrupted by focusing the scope of the inspection on the most likely and high consequence damage modes. Following the inspection, bridges that exhibit deteriorating conditions are reassessed to determine a new occurrence factor and a new inspection interval. Once established, consequence factor typically remains constant throughout the life of the bridge because the worst case scenario remains constant.

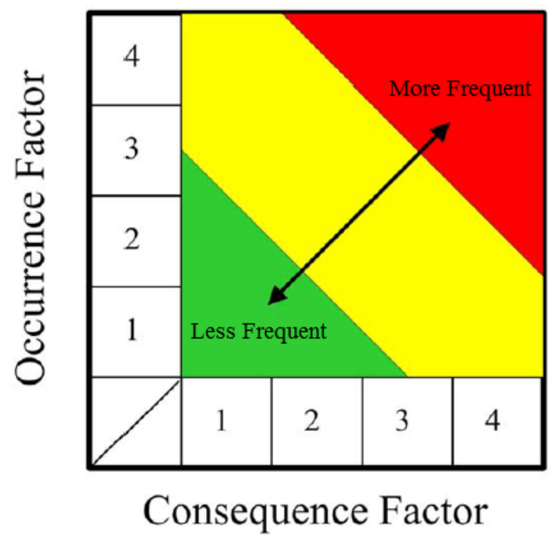

Figure 3.2: **Generic Risk Matrix for Determining Inspection Intervals**

3.1.1. Reliability Theory

Reliability is defined as the ability of an item to operate safely under designated operating conditions for a designated period of time. For bridges, reliability is a function of deterioration and accumulated damage and typically decreases as a function of time. Corrosion of steel elements is an example where deterioration increases as time progresses. Therefore, the reliability of a bridge or bridge element can be expressed as:

$$R(t) = Pr(T \geq t)$$

Equation 3.1: Reliability of a bridge element

Where R(t) is the reliability, T is the time to failure for the item, and t is the designated period of time for the item's operation. Reliability is the probability (Pr) or likelihood that the time to failure exceeds the designated operation time (Washer & Connor, 2014). The previous expression can be rearranged by substituting a probability density function that express time to failure as a distribution such as normal, log normal, etc. (Frangopol et al., 2001). A probability of failure function is time dependent and can be expressed as F(t). Then, the reliability can be expressed as:

$$R(t) = 1 - F(t)$$

Equation 3.2: Time dependent reliability

The reliability is the probability that the item will not fail during the designated operating time (Washer & Connor, 2014). A major challenge arises when applying this theory to bridge inventories. Insufficient data exist to develop a verifiable probability density function for bridges which includes all the factors that contribute to deterioration. Designs, construction practices, and environments vary widely, and performance characteristics are constantly evolving. Constant evolution makes it difficult to create a function that works for future bridges as well as older bridges because past performance may not be indicative of future performance. In addition, bridge failures are rare. This limits the quantity of available data. Further, many researchers point to the data available from laboratory tests of components to determine *strength* for attempting to set inspection intervals and develop probability density function for bridges. In reality, strength failures are not usually the issue. By illustration, a simple span bridge failing at midspan due to insufficient strength or overload is only one reason a bridge is inspected. Rather, long-term corrosion of the girder near a leaking joint is much more likely to become an issue and result in damage.

To solve these challenges, the probability of failure is determined based on qualitative or semi-quantitative analysis. Engineering judgment and experience can be used to estimate the expected reliability of a specific bridge in a given environment. Expert elicitation and expert judgment can be utilized to make decisions on reliability for complex systems subjected to complex working environments where little data are available (API, 2002). Risk can also be evaluated, as risk can be defined as the likelihood of failure during a given time interval, which is essentially the inverse of reliability. In the proposed methodology, experts estimated the expected likelihood of failure for bridge components over the time period of 72 months based upon expert judgment and the perceived likelihood of failure.

3.1.2. Definition of Failure

An important step in determining the risk of a bridge element is describing a suitable definition for "failure." For bridges, catastrophic collapse is an obvious definition; however, such collapses are rare. Therefore, a definition that captures the structural capacity, serviceability of the bridge, and the safety of the traveling public is essential.

Failure is defined as an element that is no longer performing its intended function to safely and reliably carry normal loads and maintain serviceability (Washer & Connor, 2014). To incorporate this definition with the inspection process, an element was defined to have failed when it reached the NBIS condition rating of 3, or "serious condition". (FHWA, 1995) Bridge elements in this condition may not be performing as designed. The subjective condition rating of 3 is defined as follows (FHWA, 1995):

> *NBIS Condition Rating 3: Serious Condition: Loss of section, deterioration, spalling, or scour have seriously affected primary structure components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.*

Inspectors and bridge owners are familiar with the condition rating descriptions. Because the condition ratings have been used for over forty years, there is past experience for bridges of varying designs, materials, and environments. It is not expected that bridges assessed using a risk-based approach would be allowed to reach condition rating 3. Rather, bridges would be repaired or replaced as needed to ensure the likelihood of failure remains low for the determined inspection interval. This approach may be revised slightly when element level data are used. Elements could be repaired or replaced as needed to maintain acceptable risk levels. However, the overall concept remains the same.

3.1.3. Lifetime Performance Characteristics

In a risk assessment, typical overall lifetime behavior of bridge components is important to understand. Typically, failure patterns form a “bathtub” curve with three distinct regions: infant mortality, useful life, and wear-out. Figure 3.3 provides a simplified illustration of this pattern. Different bridge components will have different shapes and timelines for their bathtub curves based upon design characteristics, construction quality, environment, and maintenance practices etc. The infant mortality section of the figure relates to the effects of construction errors or flaws that become apparent early in the life of the bridge. Components with defects typically have a shorter than expected service life and may have an increased likelihood of failure. During the useful life portion of the curve, bridge components typically have long service lives where failures are rare. The likelihood of failure within this region is low. As bridge components reach the end of their useful lives and exhibit advanced deterioration, the risk decreases and the failure rate increases. Bridges in the wear-out part of the curve require more frequent inspections to maintain adequate risk levels and to address repair needs. One goal of risk-based inspection is to extend the useful life interval by replacing or repairing bridge elements before failure. Extension of the useful life interval can also optimize inspection resources by requiring fewer inspections to maintain risk levels.

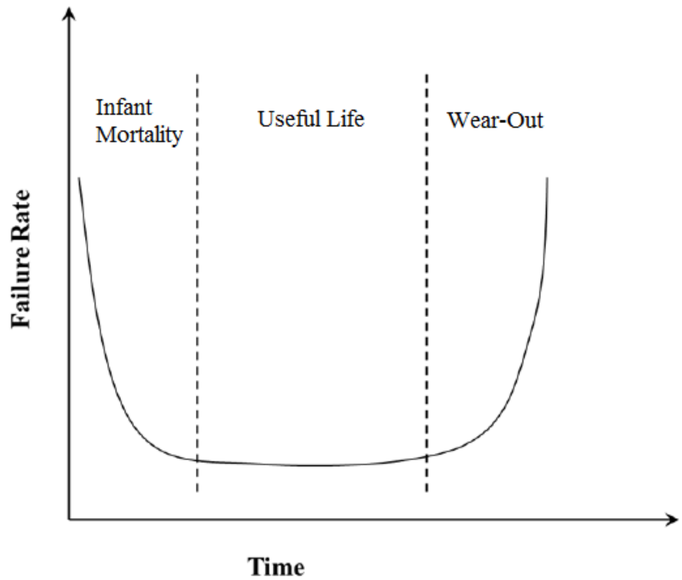

Figure 3.3: Typical Lifetime Performance Probability Curve for Highway Bridges. Adapted from NCHRP 12-82 *Developing Risk-Based Bridge Inspection Practices*.

3.2. Risk Assessment Panel

The risk assessment panel (RAP) is an expert panel assembled at the owner level to conduct analysis to support risk-based inspection. The panel assesses the reliability characteristics of bridges within a particular operational environment for a particular time interval, and the potential consequences of damage. Owner level input is important because performance characteristics of bridges and bridge elements vary widely across the United States and even within a state. Environmental conditions have a significant effect, since regions with significant snowfall apply deicing chemicals frequently and arid regions rarely apply deicing chemicals. Design and construction specifications also vary between states. Examples of these details include drainage features, use of overlays, use of protective coatings, sealers for concrete, and girder spacing. Maintenance practices also vary. As a result, knowledge and expertise of the historical performance characteristics, operational environment, design requirements, and bridge management and maintenance practices are critical for conducting risk-based assessments.

Risk-based assessments of inspection needs require expert knowledge from multiple bridge related areas. The panel of experts, or RAP, typically consists of an inspection team leader, inspection engineer, bridge program manager, structural engineer, materials engineer, academics, and outside consultants. Inspection team leaders or engineers provide insight into inspection procedures and practices. A structural engineer details common load paths and overall structural behavior, while the materials engineer weighs in on materials quality issues or material deterioration. Academics fill gaps in technical knowledge or provide independent review. Consultants supplement the knowledge base and bring an outside perspective on bridge design and inspection. With expert input from the RAP, reliability characteristics and consequences of bridge failure can be effectively assessed for a given time interval.

Expert elicitation is the method used to determine the probability or likelihood of failure of a bridge component and the associated consequence factor. The process to elicit expert judgment from the RAP consists of four parts. First, a problem statement is objectively posed to the RAP that includes basic data about the bridge. Then, each expert is asked to determine either damage modes, attributes, or the consequence factor for the presented scenario. Experts compare results and reasoning, and come to a consensus on credible damage modes, most important attributes, and consequence factor. Finally, the consensus decision and rationale is documented. If a consensus is not reached, additional information may be requested, and all sides of the discussion can be recorded for future reference. This process provides a framework for efficient, objective analysis that allows judgments from all RAP members to be considered.

3.3. <u>Occurrence Factor</u>

The occurrence factor is an expression of the probability of failure for a bridge component. The likelihood of severe damage occurring is estimated over a specified time interval and considers the likely damage modes, deterioration mechanisms, and bridge attributes. Qualitative and quantitative categorizations of the occurrence factor as well as assessment methods are presented.

3.3.1. Categorization

The estimate of probability of failure for a bridge component is expressed as the occurrence factor. To develop the occurrence factor, three factors were considered: the practical definition of failure, the time intervals for the assessment, and the required resolution of the result. In addition, an associated quantitative rating was established.

A bridge component in condition rating CR 3 is considered failed for the risk assessment. Bridge components in this condition may not be performing as designed and may exhibit severe deterioration. Linking the definition of failure to a well-known rating assessment of the bridge allows easy integration of the risk approach and the previous biennial inspection. The goal of risk-based inspections is to prevent bridge components from reaching a failed state.

The time interval for the risk assessment was based upon prior research, deterioration models, and expert judgment. Bridges typically have long service lives because deterioration mechanisms, such as corrosion, are slow acting. Commonly available reinforced concrete corrosion models indicate that corrosion initiation occurs ten or more years after the bridge was built (Enbright & Frangopol, 1998). Once initiated, corrosion may take six to twenty years to propagate depending on the corrosion resistance of the rebar (Enbright & Frangopol, 1998). Steel corrosion models in moderately aggressive environments estimate section loss on the order of 1/16 of an inch over six years (Albrecht & Hall, 2003). Research and deterioration models point towards a six year inspection interval, and expert judgment agrees. Further, if an engineer was asked to predict if a bridge element in good condition would deteriorate to serious state in one year, the likelihood of failure would be very low since deterioration mechanisms are slow-acting. The engineer's confidence in the assessment comes from his or her experience knowing that it is unlikely for significant deterioration to occur in one year. However, if an interval of ten years was asked, the uncertainty would be much higher. While it still may be unlikely for the event to happen, the engineer's ability to predict or forecast with confidence is reduced. Therefore, NCHRP 12-82 researchers debated the interval and ultimately an interval of six years was selected. It was felt that six years provides a balance between shorter intervals where the likelihood of failure would be extremely low and the confidence of assessment is high and longer intervals where the likelihood of failure increases and the confidence in the forecast is lower.

A four category qualitative scale was developed for estimating the occurrence factor for risk-based bridge assessments. The scale can be seen in Table 3.1 and ranges from Remote, where the likelihood is extremely small and no failure is expected, to High, where the likelihood of failure is increased. Four occurrence factor categories were considered to have enough precision to ensure safety and serviceability considering a one-year required resolution of the result. The slow rate of deterioration mechanisms make more exact resolutions unnecessary. For example, an inspection interval of three years and twenty one days is impractical for inspection planning and assessment purposes. Therefore, a four category scale for occurrence factor was determined to align with the resolution required for overall inspection interval.

Table 3.1: Occurrence Factor Rating Scale for Risk-based Inspections

Level	Category	Description
1	**Remote**	Remote likelihood of occurrence, unreasonable to expect failure to occur
2	**Low**	Low likelihood of occurrence
3	**Moderate**	Moderate likelihood of occurrence
4	**High**	High likelihood of occurrence

In some cases, expert judgment is quantitative in nature. Linking the qualitative and quantitative descriptions for the occurrence factor provides a common language for engineering estimates. The values shown in Table 3.2 are target values that can be used to correlate qualitative and quantitative data. Existing industrial approaches were considered when determining quantitative values. For example, the American Society of Mechanical Engineers (ASME) uses a three level scale where “low” risk has less than a 1/10,000 annual failure probability, moderate risk has an annual failure probability of 1/10,000-1/100, and high risk has an annual failure probability of greater than 1/100 (ASME, 2007). Variation and uncertainty in design, construction methods, and environment for bridges make the quantitative likelihood an order of magnitude estimate. Estimates are typically conservative, especially for less likely events.

Table 3.2: Occurrence Factor Qualitative and Quantitative Descriptions

Level	Qualitative Rating	Description	Likelihood	Expressed as a Percentage
1	**Remote**	Remote likelihood of occurrence, unreasonable to expect failure to occur	≤1/10,000	0.01% or less
2	**Low**	Low likelihood of occurrence	1/1,000-1/10,000	0.1% or less
3	**Moderate**	Moderate likelihood of occurrence	1/100-1/1,000	1% or less
4	**High**	High likelihood of occurrence	>1/100	>1%

3.3.2. Method of Assessment

The occurrence factor is assessed based upon the RAP developed damage modes and attributes. Credible damage modes are established to determine what can go wrong for various bridge components. Design, condition, loading, and screening attributes of the various components that contribute to the damage modes are determined, and ranked according to importance. Based upon this ranking, a scoring system is utilized to determine the occurrence factor.

3.3.2.1. Damage Modes

Damage modes are the answer to the question of "what can go wrong?" Forms of deterioration observable in a bridge are damage modes and can include spalling and cracking in concrete as a result of corrosion, or section loss in steel elements. Credible damage modes are determined by the RAP and are generally well-known by bridge engineers. For example, a steel girder bridge over a waterway can have damage modes of corrosion, fatigue, and fracture. Additional damage modes for consideration could be overload or impact. In this scenario, the RAP may determine that impact is not likely for a bridge over a waterway, and if the bridge is not expected to carry permit loads, overload may also not be a credible damage mode. The rate of progression for a damage mode is largely dependent upon the bridge attributes. Bridges in aggressive marine environments would be expected to corrode faster than bridges in arid environments. To answer the follow-up question of "how likely are things to go wrong?" attributes correlating to damage modes are assessed.

3.3.2.2. Attributes

Bridge component characteristics, known as attributes, affect the reliability and durability of the bridge as a whole. Attributes that enhance the reliability and durability are considered to be favorable, while attributes that decrease the reliability are considered to be unfavorable. For example, a concrete bridge deck located in a mild climate with adequate concrete cover and epoxy coated reinforcement is unlikely to experience severe damage over the inspection interval because the attributes are known to provide resistance to corrosion. In contrast, a concrete deck with minimal concrete cover and non-epoxy coated reinforcement located in a region that applies de-icing chemicals would be more likely to develop serious damage from corrosion because experience suggests those attributes are susceptible to corrosion damage. Bridge attributes are grouped into four categories: design, loading, condition, and screening. Key attributes are identified and used to assess the occurrence factor.

Design attributes describe the design of the bridge components and include items such as year of construction and concrete cover. The design attributes of a bridge frequently remain constant throughout the lifespan of the structure. Some design attributes are not recorded in the current inspection reports; however, bridge plans can supply additional information.

Loading attributes describe loads that are applied to the bridge components and include structural loading, traffic loading, and environmental loading. Examples of loading attributes include likelihood of overload, average daily truck traffic, and exposure environment. Exposure environment can be a macro concern, such as geographic region, or a local concern, such as application rate of de-icing chemicals.

Condition attributes describe bridge component conditions that are indicative of future reliability. Joint condition, presence of spalling, and shear cracking are examples of condition attributes. In general, components in deteriorated conditions are considered to be less reliable.

Screening attributes are used to identify bridges that have advanced deterioration or are outside the scope of the developed analysis. Typically, attributes that make the likelihood of serious damage very high or uncertain are considered screening attributes. Additionally, bridges with different anticipated deterioration patterns are screened out for individual consideration. Examples of screening attributes include fire damage, active fatigue cracks, and bridges with timber decks. The likelihood of serious damage resulting from fire damaged bridges is uncertain. Damage may be hidden, manifest at a future time, or may not exist. Active fatigue cracks make the likelihood of serious damage very high. It is expected that fatigue cracks can lead to fracture of the girder. Bridges with timber decks are expected to deteriorate differently than concrete decks. Therefore, timber decks that are screened from the inventory can be assessed individually. Screening attributes can facilitate effective and efficient risk rating.

The occurrence factor is evaluated by identifying key attributes and using a scoring procedure. Attributes considered by the RAP to have a major role in determining reliability of a component could be assigned a maximum of score of 20 points, and attributes that have a moderate role could be assigned a maximum 15 points. A maximum of 10 points could be awarded to attributes that play a minor role in determining reliability. Different conditions would be scaled appropriately. For example, if joint condition was considered to be a major attribute, a leaking joint could score 20 points. Debris-filled joints could score 15 points while a non-leaking joint may score 5 points. Bridges without joints could be assigned 0 points. Occurrence factor can be determined from this systematic scoring approach. This basic scoring methodology can also be customized to meet the needs of different bridge inventories.

3.4. Consequence Factor

Consequence factor is a categorization of the likely outcome determined by assuming that a damage mode results in failure of a bridge component. Based upon the likely outcome, the bridge component is placed into one of four consequence categories. Table 3.3 provides a brief summary of each category. Failure of a component is not an expected event when using a risk approach; rather, the worst-case scenario is considered to rank the importance of a given component relative to other components. When assessing the Consequence Factor, the immediate and short-term outcomes, or the results of the failure of an element should be considered. Immediate outcomes typically correlate to the safety of the bridge and surrounding public while the short-term outcome typically refers to the serviceability of the bridge and the effect on the traveling public. Factors to consider when assessing consequence factor are addressed. Detailed descriptions of each consequence category are also described. Appendix B contains additional guidance for assigning consequence factors for the deck, superstructure, and substructure.

Table 3.3: Consequence Category Brief Description

Level	Category	Consequence Description
1	**Low**	Minor effect on serviceability, no effect on safety
2	**Moderate**	Moderate effect on serviceability, minor effect on safety
3	**High**	Major effect on serviceability, moderate effect on safety
4	**Severe**	Major effect on safety and serviceability

3.4.1. Immediate Consequence

The immediate consequence refers to the structural integrity and safety of traveling public when the failure occurs. Considerations include whether a bridge will remain standing and whether the traveling public will remain safe. For example, failure of a load bearing member in a multi-girder redundant bridge is not expected to cause loss of structural integrity, excess deflections, or collapse. As a result, the traveling public is immediately unaffected when the failure occurs. A contrasting scenario would be for a fracture critical bridge, where the loss of a main member could cause excess deflection or collapse thereby causing the bridge to be immediately unsafe for the traveling public. The safety of the structure and the public should be considered for determining the immediate consequence. Spalled concrete can also create a safety issue for the traveling public by falling onto the roadway, vehicles, or property. Therefore, a concrete superstructure bridge or a bridge with a concrete deck without stay-in-place forms will have a higher immediate consequence if the bridge is over an interstate versus a bridge over a non-navigable waterway. The primary considerations for determining immediate consequence are structural integrity and public safety.

3.4.2. Short-Term Consequence

The short-term consequence refers to serviceability concerns and short-term impacts to the traveling public after a failure occurs. Load posting, repairs, and speed reductions can be considered serviceability concerns. Lane, sidewalk, or shoulder closures as a result of the damage mode impact the traveling public and can cause delays. For example, a multi-girder redundant bridge that experiences the loss of a load bearing member is expected to remain standing; however, once the failure is discovered, a typical response is to close a lane or shoulder until the bridge is repaired. Therefore, the traveling public will be affected. The effect of a lane closure for a bridge carrying an interstate will have a higher short-term consequence than a rural bridge carrying a low traffic volume. Additionally, lane closures or speed reductions for bridges located in downtown regions or bridges that are critical links to towns can cause a large impact on traveling public. The primary considerations for determining short-term consequence are serviceability concerns and impacts to the traveling public.

3.4.3. Factors to Consider

Multiple criteria exist for determining the immediate and short-term consequence factor. For some bridges, the consequence factor is clear, but for other bridges in-depth consideration is required. Some factors to consider when determining the consequence factor are:

- *ADT/ADTT*: Closing a lane or shoulder on a bridge with high ADT may result in longer queues and therefore longer delays to drivers than a closure for a bridge with low ADT. Generally, damage to bridges with high ADT will have a greater consequence factor than damage to bridges with low ADT.

- *Feature Under:* The feature under a bridge determines the immediate consequence for falling debris from the bridge. Falling debris from a bridge over a traveled roadway or walkway would have a higher consequence than falling debris from a bridge over an unpopulated area. The feature under also determines the short-term consequence if a lane or shoulder is closed to facilitate repairs.

- *Feature Carried*: Interstates often have different lane or shoulder closure policies than state highways. Consequence factor could be correlated to functional classification based upon the perceived roadway importance and effect on the traveling public.

- *Stay-in-Place Forms*: Bridges without stay-in-place forms for the underside of the deck pose a safety issue to the public beneath the bridge. Spalled concrete may fall onto the traveling public and create a safety concern. Bridges with stay-in-place forms may prevent spalled concrete from reaching the traveling public beneath the bridge.

- *Redundancy:* Non-redundant bridges are expected to have structural integrity issues should loss of a load bearing member occur. Redundant bridges behave differently than non-redundant bridges and are expected to maintain structural integrity should loss of a loading bearing member occur. Member, load path, and structural redundancy should be considered.

- *Composite Action:* Beams in non-composite bridges have the possibility of falling from the bridge in the event of failure. This creates safety and serviceability concerns for the traveling public. For composite bridges, this is expected to be a non-issue.

- *Load Carrying Capacity*: A bridge that has been previously load posted may not respond in the same manner as a bridge rated at full capacity if failure were to occur.

3.4.4. Consequence Factors

There are four consequence factor categories: Low, Moderate, High, and Severe. A general description, samples situations, and additional commentary for each scenario is presented. Tables created during the course of this research elaborating on consequence factor guidance for each bridge component can be found in Appendix B.

3.4.4.1. Low Consequence

This scenario is the least serious of all the Consequence Factor categories. The likelihood of structural collapse resulting from the damage mode is not credible and any effect on the serviceability of the bridge is minor. In order to select the lowest consequence category, the user must be able to clearly demonstrate that the consequence of the damage will be benign. Generally speaking, this decision will most often be based on engineering judgment and experience. Situations where selection of this consequence scenario may be appropriate are as follows:

- Failure of a deck overlay
- Spalling in a concrete deck bridge on a low volume and/or low speed roadway
- Spalling/corrosion damage in an abutment where the bridge is over a non-navigable waterway or unused right-of-way land.

3.4.4.2. Moderate Consequence

This scenario can be characterized by consequences that are classified as moderate in terms of their outcome. The likelihood of collapse and loss of life is very remote, and there is a minor effect on the safety of the traveling public. In order to classify the consequence of a given failure scenario as moderate, the user must demonstrate that the damage mode will typically result in a serviceability issue. The damage mode poses no serious threat to the structural integrity of the bridge or to the safety of the public. Generally, damage that will require repairs that can be addressed in a programmed fashion (i.e., non-emergency), would be classified as having a moderate consequence. Member or structural redundancy should be a consideration, and in cases where the member is non-redundant, it may be practical to classify an event higher in consequence. Situations where the selection of this Consequence Factor may be appropriate are as follows:

- Spalling damage in a deck soffit or concrete girder for a bridge over multi-use path, railroad, or low volume (<10 ADT) roadway
- Spalling in a concrete deck bridge on a moderate volume roadway
- Lane or shoulder closure on a bridge carrying a moderate volume urban roadway or a high volume rural roadway that would cause moderate delays for drivers
- Fatigue cracks that require repair but are not the result of primary member stresses, such as out-of-plane distortion cracks in redundant members

The examples above illustrate some of the element failure scenarios that would typically be categorized as having moderate consequence. In some cases, failure scenarios that could be considered more serious can be categorized as having moderate consequences, if analysis or past experience can be used to better define the outcome of a given scenario. For example, out-of-plane fatigue cracks are not uncommon in older steel bridges, and are included in the examples above. However, other types of fatigue cracks may be more serious. Cracking in a single plate of a built-up riveted girder would normally be expected have a High or Severe consequence factor if it is assumed that the crack propagates such that the load carrying capacity of the girder is lost. However, in many cases, riveted built-up members are comprised of two or three cover plates, two angles, and the girder web. If analysis showed that even after complete cracking of one of these individual components (e.g., complete cracking of one of the cover plates) the member still has reserve capacity, then it might be reasonable to classify the event as a

Moderate consequence scenario. Current load rating, overall system redundancy, and other factors should influence the decision as well. If experience and judgment are used to determine consequence factor, sufficient documentation would need to be available to justify the selection of a given consequence factor.

3.4.4.3. High Consequence

This scenario can be characterized by consequences that are more serious in terms of their outcome. The likelihood of collapse and loss of life may be more measureable, but is still relatively remote. Though the bridge may require repairs, the outcome would not be catastrophic in nature. Examples of high consequence events would include scenarios that require short-term closures for repairs, lane restrictions that have a major impact on traffic, load postings, or other actions that majorly affect the public. Situations where the selection of this Consequence Factor may be appropriate are as follows:

- Failure of a main member in a multi-girder bridge with sufficient load path redundancy
- Spalling damage in a deck soffit or concrete girder for a bridge over a navigable waterway or a moderate/high volume roadway
- Spalling in a concrete deck bridge on a high volume roadway
- Lane or shoulder closure on or under roadway that would cause major delays for drivers
- Impact damage on a multi-girder bridge

Using brittle fracture of an exterior steel girder in a multi-girder bridge as an example, the immediate consequence is assumed to be High. Structural capacity is expected to remain adequate based upon experience. If engineering calculations were performed that quantifiably showed that the bridge had sufficient reserve capacity in the faulted condition, i.e. with one girder fractured, it might be reasonable to identify the event as having a Moderate immediate consequence. The short term consequence would be dependent upon site conditions at the bridge including traffic volume, feature carried, and feature under.

3.4.4.4. Severe Consequence

This is the most critical consequence factor category and can be characterized by events that, should they occur, are anticipated to result in catastrophic outcomes. Structural collapse and loss of life are likely should the failure occur. Because of the catastrophic nature implied by this consequence scenario, it should not be selected arbitrarily as a catch-all or just to be conservative. Examples of severe consequence events would include failure of the pin or hanger in a bridge with a suspended truss span or a two girder system, or strand fractures in a pre- or post-tensioned element that results in a non-composite member falling into a roadway below. Situations where the selection of this Consequence Factor may be appropriate are as follows:

- Fracture in a non-redundant steel bridge member
- Failure of a non-composite girder over traffic
- Spalling of a concrete soffit, concrete girder, or concrete abutment over a high volume roadway or pedestrian walkway
- Lane or shoulder closure on a major roadway that would cause significant delays for the traveling public
- Bearing area failure resulting in deck misalignment

Cases where there is insufficient experience or where reliable calculations cannot be made may also be categorized as severe. Examples would be unique, one-of-a-kind bridges or other structural systems where the result of failure associated with a given damage mode is essentially unknown. In such cases, the only reasonable approach is to assume and select a Severe consequence, as the actual outcome cannot be well defined.

Downgrading to a less serious consequence factor is permitted but only through the use of analysis. Experience alone may not be used to justify downgrading from a

Severe consequence to a High consequence, due to the catastrophic outcomes associated with the more severe scenario. While experience may be used in conjunction with analytical studies to make a stronger case for downgrading to a lower consequence scenario, experience alone is not deemed to be sufficient.

3.5. Inspection Procedures

The inspection process plays a key role in updating the information used in the risk assessment. Specific information regarding the current condition of the bridge elements is critical for determining the occurrence factor. For example, to determine the appropriate occurrence factor for corrosion damage in a steel beam, information on the current extent of corrosion damage is needed to assess whether severe damage is likely to develop over the inspection interval. To gather information for the assessment, visual inspections are typically adequate, although non-destructive evaluation or a hands-on inspection may be required. In some cases, the RAP may specify the type of inspection required to obtain the necessary data. During the inspection, additional information about the bridge, such as concrete cover, may need to be collected to ensure an accurate risk assessment. Also, data such as the presence of spalling may need to be refined to fit into categories such as “greater than 20% spalled by area”, or “less than 5% spalled by area”. Similar reporting styles and classifications would create consistency between data gathered from inspections and therefore greater consistency in the risk assessments.

Currently, there are a variety of approaches used by different states to collect, document, and store bridge inspection data. Some states use the component-based system mandated by the NBIS; others use a span-by-span approach; and others use an element-level process. Element-level inspections lend themselves to risk-based inspections. Element-level inspections collect more detailed and descriptive information than component-level or span-by-span inspections. Information needed to support a risk assessment includes the key damage modes affecting elements of the bridge, the location and extent of damage, and the condition of key attributes developed during the RAP meeting. An element-level inspection addresses all of these needs and relates inspection to the data needed for the assessment.

Inspections can be prioritized based upon the Inspection Priority Number, or IPN. The IPN is the product of the occurrence factor (O) and the consequence factor (C):

$$IPN = O \times C$$

Equation 3.3: Inspection Priority Number

The IPN highlights the damages modes that have the highest likelihood of failure and the greatest associated consequence (Washer & Connor, 2014). This information can allow bridge inspectors to emphasize certain elements during the inspection based on the engineering analysis and rationale developed by the RAP. However, the scope of the inspection is not limited to the damage modes that have the highest IPN. Other elements should be inspected to ensure the validity of the current occurrence factor assessment or to determine if additional deterioration warrants a change in the occurrence factor assessment. Overall, the IPN allows a more focused inspection based upon the engineering assessment of the specific bridge and improves the effectiveness of the inspection.

3.6. Summary

The risk methodology was created to ensure bridge safety, optimize the inspection process, be easily implemented, meet the needs of different states, and utilize existing knowledge of in-service bridge behavior. Key elements of the methodology included expert input from the risk assessment panel (RAP), determination of the occurrence factor, determination of the consequence factor, and inspection procedures. A semi-quantitative risk-based framework was developed by the RAP for the risk assessment. Occurrence factor had four qualitative categories, as did the consequence factor. Consequence factor was determined through immediate and short-term consequence scenarios. Element-level inspection approaches were best suited for risk-based assessments. Overall, risk-based inspections appear to be a viable method to ensure bridge safety and optimize the inspection process.

CHAPTER 4. INDIANA RISK ASSESSMENT PANEL MEETING

The Risk Assessment Panel (RAP) meeting was a consensus-based expert elicitation approach utilized to develop and refine data models created in the NCHRP 12-82 study for the risk-based inspection approach. Expert elicitation aims to quantify the likelihood of adverse future events when insufficient operational data exists to make a quantitative estimate. With input from the expert members of the RAP, a comprehensive and objective framework was developed for determining bridge inspection intervals. Because each state operates using different design and construction specifications and has different environmental conditions, expert opinions formed at the owner level allow the RBI process to be customized for each state. The results from the Indiana RAP meeting are presented below.

4.1. Meeting Overview

The RAP meeting consisted of a two-day event at the Indiana Department of Transportation (INDOT) office in Indianapolis, Indiana. To begin the meeting, the goals, objectives, and overall research approach were described. Examples of applications for the approach were also presented as a training exercise for the occurrence factor and consequence factor. During the expert elicitation section of the meeting credible damage modes for concrete decks, steel superstructures, and prestressed concrete superstructures were identified through the consensus of the RAP. Time restraints prevented concrete superstructures and substructures from being specifically addressed in this meeting. Additional meetings could be held prior to complete implementation of the methodology. Also, relevant attributes were identified and ranked in order of importance. This ranking (high, medium, or low) was used to establish a preliminary scoring method.

Consequence factors for each bridge type were also developed through the consensus based expert elicitation approach. After the meeting, the information was analyzed and organized into scoring models. Back-casting, which involved monitoring deterioration progression through historical data and comparing the results with the risk approach, was used to evaluate the safety and effectiveness of the RAP specific risk model.

4.1.1. RAP Meeting Attendees

The Indiana RAP Panel was attended by INDOT officials, industry consulting experts, and officials from the Federal Highway Administration. Twelve participants were present for the first day of the meeting, and nine were present the second day. A listing of RAP meeting attendees is available in Table 4.1.

Table 4.1: Listing of RAP Meeting Attendees

Name	Current Position	Affiliation
Participant A	Director of Bridges	INDOT
Participant B	Bridge Inspection Engineer	INDOT
Participant C	Structural Services	INDOT
Participant D	Bridge Inspection Manager	INDOT
Participant E	Program Engineer	INDOT
Participant F	Structural Services	INDOT
Participant G	Bridge Standards and Policy Engineer	INDOT
Participant H	Senior Project Manager	Beam, Longest & Neff
Participant I	Senior Project Manager	United Consulting
Participant J	Steel Bridge Design Engineer	FHWA
Participant K	Bridge Engineer	FHWA
Participant L	National Bridge Inspection Program Engineer	FHWA

4.1.2. Schedule and Agenda

Discussion on the first day of the workshop centered on understanding the RBI approach and determining likelihood for multiple bridge components. PowerPoint presentations introduced the objectives and goals of the meeting as well as presented three examples that were used as training. After training, participants listed and came to a consensus on damage modes for decks and steel superstructures for bridges in Indiana. The panel also determined attributes for each of the damage modes and ranked them according to importance.

Discussion on the second day of the workshop centered on prestressed superstructures and consequence analysis. Damage modes and attributes were determined for prestressed superstructure bridges in Indiana using the consensus approach. Then, consequence factors for deck, steel, and prestressed damage modes were categorized. At the end of the meeting, Indiana specific criteria for risk-based inspections had been established.

4.1.3. Expert Elicitation Process

The process to elicit expert judgment from the RAP has four major components: statement of the problem, expert elicitation, comparison of results, and documentation.

1. *Statement of the Problem*: The RAP was presented with a problem statement that included data such as bridge design, location, and traffic patterns. The problem statement was phrased to avoid biased decisions, and damage modes were determined directly from the statement. To determine the attributes, a damage mode was assumed, and to determine the consequence factor, a given failure scenario was assumed.

2. *Expert Elicitation*: Each member of the RAP was asked to independently determine damage mode, attribute, and consequence factor based upon his or her judgment, experience, available data, and the scenario presented. The expert provided an assessment in ten percent increments of likelihood or consequence.

3. *Comparison of Results*: Each expert shared his or her results and reasoning, and the results were compared. In many cases, a consensus was reached. If a consensus was not immediately apparent, experts had the opportunity to discuss

the various judgments and revise their scores. If a consensus was still not reached after discussion, the most conservative factor was adopted.

4. *Documentation*: The results from the expert elicitation were documented. For items where a consensus was reached, rationale for making the determination were noted. If a consensus was not reached, all facets of the discussion were recorded for future reference. Additional supporting information was also included in the INDOT bridge file as needed.

4.1.3.1. Identifying Damage Modes

Experts utilized a blank worksheet similar to Table 4.2 to determine damage modes for bridge elements. After listing credible damage modes individually, each expert shared his or her judgments with the rest of the panel. Most damage modes were well known by the experts and bridge engineers, and a consensus on the most likely and least likely damage modes was quickly and easily reached. This method allowed states to address damage modes that may be a specific concern for their inventory. In addition, the process acted as a filter that allows the most important and likely damage modes to rise to the top while the less likely or unrealistic damage modes fall to the bottom of the list. Another beneficial aspect of this approach was that identical elements have identical damage modes, e.g., steel girders exhibit the same damage modes. This allowed additional assessments to be completed quickly and efficiently.

Table 4.2: Expert Elicitation for Steel Girder Damage Modes

Damage Mode	Likelihood (in 10% increments)
Corrosion / Severe section loss	● ● ● ● ● ○ ○ ○ ○ ○
Fatigue Cracking	● ● ○ ○ ○ ○ ○ ○ ○ ○
Impact damage/ Fire	● ● ○ ○ ○ ○ ○ ○ ○ ○
Overload	● ○ ○ ○ ○ ○ ○ ○ ○ ○
Stress Corrosion Cracking	○ ○ ○ ○ ○ ○ ○ ○ ○ ○
	10% 20% 30% 40% 50% 60% 70% 80% 90% 100%

As an example, to determine the damage modes for a steel girder, the following question was posed to the experts: "The current condition rating for a steel girder is a three, serious condition. What damage mode is likely to be present?" Table 4.2 shows the results from one expert. He believed that corrosion and section loss were the most likely damage modes that would contribute to the deteriorated condition. Overload was a possible damage mode, but less likely to occur, and stress corrosion cracking was identified but not assigned any likelihood, indicating he believed the chance of occurrence was less than ten percent. Ultimately, the consensus of the panel was that corrosion was the most likely damage mode for steel girders.

4.1.3.2. Identifying Attributes

To determine the attributes associated with each damage mode, experts individually listed attributes that would indicate a given damage mode was likely to occur. A group discussion was then conducted to determine key attributes and their relative importance. Attributes were ranked according four categories: high, medium, low, and screening. Screening criteria were used to identify bridges with known issues or atypical bridges that required further analysis. Attributes ranked as high had the greatest influence on the damage mode, while attributes ranked as low had the least influence. The occurrence factor was then determined from a scoring procedure.

Figure 4.1: RAP Determined Attributes for Section Loss on Steel Girders

Continuing the example of a steel girder with the damage mode of corrosion/section loss, the panel was asked, "For the steel girder, what information would be needed to make the assessment of how long before corrosion/section loss becomes an issue?" As shown in Figure 4.1, the panel suggested the most important attributes were the condition of the expansion joints, the environment, and the presence of existing section loss. These were ranked as high and assigned the maximum possible point values. Medium level attributes included maintenance practices, structure age, and coating type and were assigned fewer possible points. Attributes ranked as low were assigned the least amount of possible points. Based upon these distinctions, a basic scoring procedure was developed and used to determine occurrence factor.

4.1.3.3. Identifying Consequence Factors

Experts utilized the blank worksheet seen in Table 4.3 to identify consequence factors. Low, Moderate, High, and Severe were the consequence categories utilized. An exercise was conducted in which the question was asked, "Providing that the damage mode occurred, what is the consequence?" Experts would then fill in the bubbles in the form based upon their judgment and experience, and a discussion would follow. Consensus on the appropriate consequence factor was typically reached during the discussion. There were six scenarios presented to the panel for evaluation ranging from the loss of a load bearing girder to spalling in the substructure. This process was used to determine consensus, to address situations where there was disagreement, and to discover unique situations that may require further expert judgment.

Table 4.3: Worksheet used to identify consequence factors

Consequence Category	(%)
Low	○○○○○○○○○○
Moderate	○○○○○○○○○○
High	○○○○○○○○○○
Severe	○○○○○○○○○○

An example for determining the consequence factor is shown in Figure 4.2. For a steel girder, the evaluated damage mode was loss of capacity in one member. The red numbers in the grid represent the number of experts who recorded that answer. Four members expressed that the loss of capacity had a 30% likelihood of being a Moderate consequence. The high percentages recorded for Moderate and High demonstrated that the majority of members believed the scenario to be either a Moderate or High consequence event. Ultimately, the consensus was that losing a girder in a steel bridge was a High consequence event.

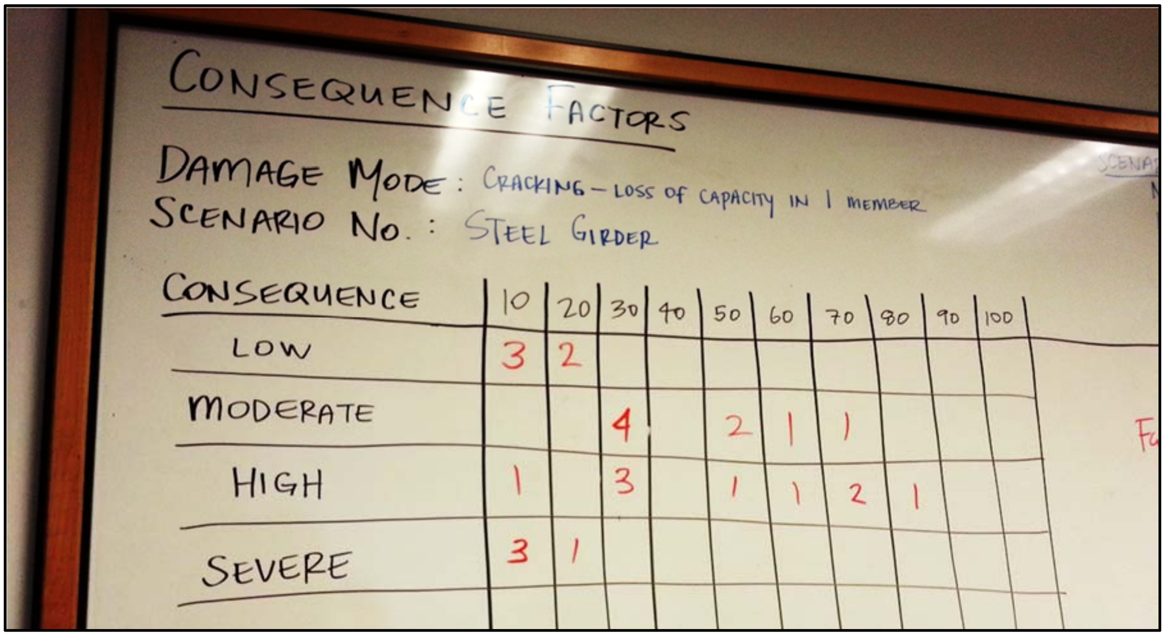

Figure 4.2: Determining Consequence Factor for Loss of Capacity in a Steel Girder

4.2. Decks

Damage modes, attributes, and consequence factors were determined by the RAP for concrete bridge decks in the state of Indiana. Corrosion was established as the primary damage mode. Attributes included exposure environment, current deck condition, and maintenance cycle. The consequence factor was assumed to vary between Low and High depending on site specific conditions at the bridge.

4.2.1. Damage Modes

RAP members listed cracking, corrosion, rubblization, rutting, and debonding as damage modes for concrete decks. After discussion, rubblization and rutting were not considered to be credible damage modes in the state of Indiana, and debonding was considered to be a damage mode for the overlay and not for the concrete deck. The remaining damage modes, cracking and corrosion, were considered to be interrelated. Therefore, the primary damage mode for concrete decks in Indiana was corrosion.

4.2.2. Attributes and Scoring

Based on the deck damage mode of corrosion, attributes and their relative importance were determined by the RAP panel. These are summarized in Table 4.4 and were integrated into the 12-82 risk framework. If the attribute was similar to an item presented in the NCHRP 12-82 study, it was noted. The high, medium, low, and remote columns distinguish how the points would be awarded for a given attribute. If the bridge exhibited the condition shown in the high column, maximum points would be awarded. Current deck condition, maintenance cycle, and exposure environment were agreed to have a high degree of severity. The degree of severity and max score columns correlated the RAP consensus importance with the points assigned to that attribute. For example, the attribute "current deck condition" has a high degree of severity, and was assigned a maximum point value of twenty points. A bridge deck in condition rating five or below received twenty points. Decks in condition rating six received five points, and a condition rating of seven or above received zero points.

Four screening attributes were identified: bridges with a non-composite superstructure, bridges with a known construction error, bridges that did not have a concrete deck, and bridge decks in condition rating CR 4 or below. Non-composite bridges can exhibit different deterioration patterns than composite bridges and have increased reliability concerns. Composite bridge decks are also preferred in current design provisions. Bridges with a known construction error behave according to the specific error. The risk procedure may not capture the unique deterioration pattern, and the extended inspection interval should be used with caution. Bridges without a concrete deck were also screened out because the majority of bridges in Indiana are concrete decks. The RAP only addressed damage modes and attributes associated with concrete decks and further RAP meetings could involve creating a scoring system for other deck types. Decks in condition rating CR 4 or below were screened out to automatically have 24 month inspection interval based upon the level of deterioration and likelihood of reaching the failure condition during the next inspection interval.

Table 4.4: Attributes for the Damage Mode of Deck Corrosion

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
C.1	Current Deck Condition	CR 5	CR 6	CR 7+			H	20
C.11	Presence of Repairs	Yes			No		L	10
C.13	Efflorescence/ Leaching	Efflorescence with rust staining	Moderate Efflor.	Minor Efflor. without rust staining	No Efflor.		M	15
	Maintenance Cycle	No maintenance			Washing / Sealing		H	20
D.11	Concrete Cover	<1.5"	1.5" - 2.5"	2.5"+			M	15
D.12	Reinforcement Type	Not Epoxy Coated			Epoxy Coated		M	15
D.4	Deck Drainage	Ponding/ Ineffective Drainage			Effective Drainage		M	15
D.7	Presence of Overlay/Type	Bituminous without Membrane			No Overlay or LMC overlay		M	15
L.1	ADTT (Functional Class)	>2500 -- Interstate			<100 -- Rural		M	15
L.3	Exposure Environment	Northern Districts	Central Districts	Southern Districts			H	20
-	Composite with Superstructure					X		
-	Construction Error					X		

4.2.3. Consequence Factor

Consequence factor was determined for the damage mode of deck corrosion by the RAP. The panel was split on whether deck corrosion had a Moderate or High consequence, as shown in Table 4.5. Discussion centered around whether the damage occurred on the top of the deck in the form of potholes, or whether the corrosion occurred on the underside of the deck as spalling. Traffic volume and feature intersected were also considerations. A bridge with potholes would have a higher consequence on a high-speed high-volume roadway than on a low-speed low-volume roadway. Additionally, spalls from the deck underside could pose a safety hazard to the public underneath the bridge. A bridge over a roadway would have a higher consequence than a bridge over a non-navigable waterway. The RAP consensus was that deck corrosion did not have an exclusive consequence factor and was largely dependent upon site specific conditions at each bridge.

Table 4.5: RAP Results: Consequence Factor for Deck Corrosion

Deck: Corrosion Consequence					
		Low	**Moderate**	**High**	**Severe**
1	Participant 1	90	10	-	-
2	Participant 2	30	40	20	10
3	Participant 3	-	50	50	-
4	Participant 4	-	10	40	50
5	Participant 5	-	40	60	-
6	Participant 6	-	90	10	-
7	Participant 7	10	50	40	-
8	Participant 8	-	40	60	-
9	Participant 9	-	20	80	-
		0.14	**0.39**	**0.40**	**0.07**
The Consequence of this Damage Mode is Moderate/High.					

4.3. Steel Superstructure

Damage modes, attributes, and consequence factors were determined by the RAP for steel superstructure bridges in the state of Indiana. Corrosion, fatigue, and fracture were established as the primary damage modes. Attributes included condition of the joints, exposure environment, and fatigue detail category. The consequence factor was assumed to be either Moderate or High depending on site specific conditions at the bridge. Fracture critical bridges were not included in the steel superstructure discussion.

4.3.1. Damage Modes

RAP members listed corrosion/section loss, impact, overload, fatigue cracking, fracture, and bearing failure as damage modes for steel superstructures. After discussion, impact damage was determined to be an attribute for fracture and not a damage mode. Bearing failure and overload were initially listed as damage modes, but RAP members considered them low likelihood events. Bearing failure and overload were therefore not deemed credible damage modes, and removed from the list. The three primary damage modes were agreed to be corrosion/section loss, fatigue cracking, and fracture.

4.3.2. Attributes and Scoring

Based on the steel superstructure damage mode of corrosion/section loss, attributes and their relative importance were determined by the RAP panel. These are summarized in Table 4.6 and were integrated into the NCHRP 12-82 risk framework (Washer & Connor, 2014). Existing section loss, joint condition, and exposure environment were determined to have a high degree of severity. The degree of severity and max score columns correlated the RAP consensus importance with the points assigned to that attribute. For example, the exposure environment was assigned a high degree of severity. Indiana climate regions can be divided into approximately three sections: the northern climate with significant snowfall and application of de-icing chemicals, the southern climate with milder winters but humid summers, and the middle climate with a combination of the two. Climate conditions are favorable for corrosion in the entire state, so all bridges were awarded points with differences based upon location. The northern climate consists of the LaPorte and Fort Wayne districts, and bridges in these regions received the maximum number of points, twenty. Crawfordsville and Greenfield are the two middle districts, and bridges in these regions received fifteen points. Bridges in the southern districts of Vincennes and Seymour received ten points.

Three screening attributes were identified: type of deck, existing section loss, and current condition rating. Bridges with timber or open decks allow water to drain on the superstructure and increase the rate of corrosion. These types of bridges would require additional analysis before a risk approach could be implemented. Additionally, bridges exhibiting advanced section loss were screened out of the inventory. The advanced deterioration was a warning sign that the bridge required extra monitoring. Steel superstructures in condition rating CR 4 or below were screened out to automatically have 24 month inspection interval based upon the level of deterioration and likelihood of reaching the failure condition during the next inspection interval.

Table 4.6: Attributes for Steel Superstructure Corrosion

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
-	Type of Deck					X		
C.17	Coating Type/ Condition	No coating/ Ineffective			Effective Coating		M	15
C.21	Existing Section Loss	Significant amount of corrosion	Moderate amount of corrosion	Minor amount of corrosion	No active corrosion	X	H	20
C.4	Adequate Drainage	Drains onto superstructure			Adequate Drainage		M	15
C.5	Maintenance Cycle	No maintenance			Regular Maintenance		M	15
C.7	Condition of Joints	Open Joints/Failed Joints	Leaky Joints	New Joints	Jointless Bridge		H	20
D.6	Year of Construction		2000 or before	2000+			M	15
L.1	ADTT (Functional Class)	>2500			<100		L	10
L.3	Exposure Environment	Northern Districts	Central Districts	Southern Districts			H	20

Based on the steel superstructure damage mode of fatigue cracking, attributes and their relative importance were determined by the RAP panel. These are summarized in Table 4.7 and were integrated into the 12-82 risk framework. Existing fatigue cracks, presence of repaired cracks, existing distortion induced cracks, fatigue detail, and average daily truck traffic were determined to have a high degree of severity. The degree of severity and max score columns correlated the RAP consensus importance with the points assigned to that attribute. For example, the fatigue detail category was assigned a high degree of severity. Bridges with fatigue detail Category E or E' were assigned the maximum point value of twenty points. A bridge with fatigue detail Category D was assigned fifteen points, and bridges with fatigue Category A, B, B' or C were assigned zero points. The Indiana RAP grouped fatigue Category C with the A and B details because experience showed that cracking in Category C details was not observed to be an issue in Indiana.

The presence of active fatigue cracks due to primary stresses was the screening attribute identified for the steel superstructure damage mode of fatigue cracking. Active fatigue cracks in elements due to primary stresses can propagate quickly and lead to failure of the steel beam. Retrofitting the fatigue cracks before using the risk method was recommended.

Table 4.7: Attributes for Steel Superstructure Fatigue Cracking

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
C.18	Existing Fatigue Cracks	Yes			No	X	H	20
C.18	Presence of Repaired Cracks	Yes			No		H	20
C.18	Existing Distortion Induced Cracks	Yes			No		H	20
D.16	Fatigue Detail	E/ E'	D		C / B / A		H	20
D.6	Year of Construction	<1975	1976-1984	1985-1993	1994+		M	15
L.1	ADTT (Functional Class)	>2500			<100		H	20

Discussion on the steel superstructure damage mode of fracture occurred; however attributes and their relative importance were not specifically determined during the RAP meeting. The primary change from the 12-82 methodology was for the attribute of average daily truck traffic. The importance was reduced from a high degree of severity to a medium degree of severity, and the maximum point cutoff changed from 1,000 ADTT to 2,500 ADTT. Bridges with over 2,500 ADTT received the maximum point score of fifteen points. The attributes with a high degree of severity were year of construction and previous impact history. Prior to 1975 a fracture control plan for bridges did not exist. Therefore, bridges built pre-1975 are potentially more susceptible to fracture, and received twenty points. Newer bridges received fewer points because material properties, such as toughness, and design methods have improved over time. One design improvement was the elimination of details susceptible to constraint induced fracture, such as those found in the Hoan Bridge. Bridges constructed after 2009 have a very slight risk of fracture and received zero points. Screening attributes for the damage mode of fracture were not identified.

4.3.3. Consequence Factor

Consequence factor was determined for the damage mode of steel girder cracking by the RAP. In the worst case scenarios, the RAP decided damage modes of corrosion, fatigue, and fracture led to the loss of one load bearing member through brittle fracture. The panel determined the consequence was High as shown in Table 4.8. Discussion centered on structural redundancy and the effect to the traveling public caused by a lane or shoulder closure. Situations where the consequence factor could be reduced to Moderate were also debated. Items of consideration were spacing of the beams, feature carried, feature intersected, engineering analysis, and past experience. Redundant bridges on rural roads over non-navigable waterways were ideal candidates to be rated as a Moderate consequence, because experience showed the structural capacity of the bridge was expected to remain adequate for service loading, and there was a small effect on the traveling public. The RAP consensus was that loss of a load bearing member has a High consequence, unless structural analysis or engineering experience allowed the consequence factor to be reduced to Moderate.

Table 4.8: RAP Results: Consequence Factor for Loss of a Steel Girder

Steel Girder: Loss of a Member Capacity Consequence					
		Low	**Moderate**	**High**	**Severe**
1	Participant 1	10	30	50	10
2	Participant 2	-	-	80	20
3	Participant 3	10	60	30	-
4	Participant 4	-	30	60	10
5	Participant 5	10	70	10	10
6	Participant 6	-	30	70	-
7	Participant 7	-	30	70	-
8	Participant 8	20	50	30	-
9	Participant 9	20	50	30	-
		0.08	**0.39**	**0.48**	**0.06**
The Consequence of this Damage Mode is High.					

4.4. Prestressed Superstructure

Damage modes, attributes, and consequence factors were determined by the RAP for prestressed superstructure bridges in the state of Indiana. Flexural/shear cracking, corrosion, and strand fracture were established as the primary damage modes. Important attributes included current condition rating, joint condition, and exposure environment. The consequence factor was assumed to be either Moderate or High depending on site specific conditions at the bridge.

4.4.1. Damage Modes

RAP members listed corrosion, bearing area damage, flexural/shear cracking, and strand fracture as damage modes for prestressed superstructures. After discussion, bearing area damage was eliminated as a damage mode because RAP members considered bearing area damage to be a low likelihood event. In Indiana, experts expect bearings to last the life of the bridge because historically, inspection intervals have not needed to be adjusted based upon bearing seat issues. The three primary damage modes were agreed to be corrosion, flexural/shear cracking, and strand fracture.

4.4.2. Attributes and Scoring

Based on the prestressed superstructure damage mode of corrosion, attributes and their relative importance were determined by the RAP panel. These are summarized in Table 4.9 and were integrated into the 12-82 risk framework. Current superstructure condition, existing corrosion damage, concrete cover, and exposure environment were determined to have a high degree of severity. The degree of severity and max score columns correlated the RAP consensus importance with the points assigned to that attribute. For example, reinforcement type was assigned a medium degree of severity. Uncoated carbon steel was determined to increase the likelihood of corrosion propagation, and received the maximum point value of fifteen. Epoxy coated reinforcement was considered to be a beneficial attribute and was assigned a point value of zero. However, epoxy coating on both the reinforcing steel and prestressing steel was considered unfavorable and would result in points being awarded.

Three screening attributes were identified: existing corrosion damage, delayed ettringite formation (DEF), and current superstructure condition rating. Bridges that exhibit advanced corrosion were screened out of the inventory because they require additional monitoring. Poor materials may have resulted in concrete that has delayed ettringite formation. While the extent of the problem in Indiana is not known, bridges that do display DEF are screened out and repaired. Prestressed superstructures in condition rating CR 4 or below were screened out to automatically have 24 month inspection interval based upon the level of deterioration and likelihood of reaching the failure condition during the next inspection interval. A potential screening criteria would be bridges with an adjacent box beam superstructure because RAP members expressed multiple inspection related concerns about aging adjacent box beams.

Table 4.9: Attributes for Prestressed Superstructure Corrosion

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
C.1	Current Superstructure Condition	CR 4 or less	CR 5/6	CR 7+			H	20
C.8	Existing Corrosion Damage	Significant amount of corrosion	Moderate amount of corrosion	Minor amount of corrosion	No active corrosion	X	H	20
D.11	Concrete Cover	<1.5"	1.5" - 2.5"	2.5"+			H	20
D.12	Reinforcement Type	Not Epoxy Coated			Epoxy Coated		M	15
D.18*	Bad End Detail	Strand Exposed to Environment			Not Exposed to Environment		L	10
L.3	Exposure Environment	Northern Districts	Central Districts	Southern Districts			H	20
--	Delayed Ettringite Formation	Poor materials			Acceptable Materials	X		

Based on the prestressed superstructure damage mode of flexural/shear cracking, attributes and their relative importance were determined by the RAP panel. These are summarized in Table 4.10 and were integrated into the 12-82 risk framework. Load posting status was the only attribute determined to have a high degree of severity. The degree of severity and max score columns correlated the RAP consensus importance with the points assigned to that attribute. Likelihood of overload had a low degree of severity, and was typically determined by identifying roads where permit loads travel. Bridges on roads that regularly carry permit loads have a high likelihood of overload and were assigned ten points. A moderate likelihood of overload consisted of bridges that occasionally carried permit loads and were given five points. Bridges that were not expected to carry permit loads received zero points. No screening attributes were identified for the prestressed superstructure damage mode of flexural/shear cracking.

Table 4.10: Attributes for Prestressed Superstructure Shear Cracking

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
D.2	Load Posting	Posted			Not Posted		H	20
D.6	Year of Construction		<2000	>2000			L	10
L.4	Likelihood of Overload	High Likelihood		Low Likelihood			L	10
C.14	Flexural Cracking	Cracks > 0.006 inches wide			No cracking		L	10

The prestressed superstructure damage mode of strand fracture was briefly discussed. Attributes and relative importance determined during the Oregon RAP meeting were presented to the Indiana RAP. Members of the Indiana RAP decided to adopt the attributes and relative importance from the Oregon RAP, with a minor change to the attribute "presence of repaired areas". Presence of repaired areas was considered to have a low degree of severity instead of a medium degree of severity. Therefore, bridges with a significant amount of repaired areas were given ten points, bridges with moderate amount of repaired areas received six points, and bridges with a minor amount of repaired areas were scored at three points. Bridges with no repaired areas received zero points. No screening criteria were established.

4.4.3. Consequence Factor

Consequence factor was determined for the damage mode of prestressed strand corrosion by the RAP. In the worst case scenarios, the damage modes of corrosion, shear/flexural cracking, and strand fracture lead to girder cracking and the loss of one load bearing member. The panel determined the consequence was High as shown in Table 4.11. Discussion was similar to the steel superstructure consequence factor.

Structural and public safety was the first consideration. Structural capacity of the bridges was expected to remain adequate. Feature under was also a consideration because spalled concrete from the prestressed beam could create a safety concern for the traveling public under the bridge. The short-term consequence and effect of a lane or shoulder closure were also studied. An Indiana document titled "Interstate Congestion Policy" clarified impacts of lane and shoulder closures based upon the roadway classification, time of day, and traffic volume. This document was recommended as an aid to assess the short-term consequence. Situations where the consequence factor could be classified as Moderate were also discussed. Similar to steel superstructure bridges, rural low-volume bridges over non-navigable waterways were ideal candidates for the reduction in consequence factor based upon engineering experience and structural analysis. The RAP consensus was that loss of a load bearing member had a High consequence, unless structural analysis or engineering experience allowed the consequence factor to be reduced to Moderate.

Table 4.11: RAP Results: Consequence Factor for Prestressed Girder Strand Corrosion

Prestressed Strand Corrosion					
		Low	**Moderate**	**High**	**Severe**
1	Participant 1	-	-	50	50
2	Participant 2	-	-	-	100
3	Participant 3	-	20	50	30
4	Participant 4	-	0	70	30
5	Participant 5	-	40	60	-
6	Participant 6	-	10	80	10
7	Participant 7	-	30	60	10
8	Participant 8	-	40	50	10
9	Participant 9	-	40	60	-
10	Participant 10	-	30	70	-
11	Participant 11	-	30	50	20
		0.00	**0.22**	**0.55**	**0.24**
The Consequence of this Damage Mode is High.					

4.5. Concrete Superstructure

The RAP did not specifically determine damage modes and attributes for reinforced concrete superstructures. However, similarities existed between steel, prestressed and concrete superstructures that allowed the risk assessment to be customized. Concrete superstructure damage modes considered were bearing area damage, corrosion between beam ends, flexural cracking, and shear cracking. Bearing area damage was viewed by the RAP to be a low likelihood event and therefore eliminated as a damage mode. Additionally, if an attribute was previously analyzed by the RAP, the attribute carried through to the rating for concrete superstructure. For example, maintenance cycle was an attribute that applied to the damage mode of prestressed superstructure corrosion. Because the concrete superstructure also had a damage mode of corrosion, the determined attribute properties were applied to concrete superstructures as well. Table 4.12 lists the attributes that were changed from NCHRP 12-82 to be consistent with previous Indiana RAP determinations.

Table 4.12: INDOT Specific Attributes for Concrete Superstructure

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	NCHRP 12-82 Degree of Severity	INDOT Degree of Severity	Max Score
C.5	Maintenance Cycle	No maintenance			Washing / Sealing	H	M	15
C.11	Presence of Repaired Areas	Significant amount of repaired areas	Moderate amount of repaired areas	Minor amount of repaired areas	No repaired areas	M	L	10
C.13	Efflorescence/ Staining	Efflorescence with rust staining	Moderate Efflor.	Minor Efflor. without rust staining	No Efflor.	H	M	15
L.4	Likelihood of Overload	High Likelihood		Low Likelihood		M	L	10

Similar to the consequence factor for prestressed superstructures, the consequence factor for concrete superstructures was High. Structural capacity was expected to remain adequate for service loads, and falling debris from the beam could affect the safety of the public beneath the bridge. Short-term consequence varied depending on feature carried and traffic volume. Structural analysis or engineering experience could allow a reduction in the consequence factor to Moderate for certain bridges.

4.6. Substructure

The RAP did not specifically determine damage modes and attributes for substructures. However, similarities existed between the previously rated bridge components that allowed the risk assessment to be customized. The damage mode considered for the substructure was corrosion. Important attributes included joint condition, exposure environment, and current substructure condition rating. Table 4.13 lists the attributes that were changed from NCHRP 12-82 to be consistent with previous Indiana RAP determinations.

Table 4.13: INDOT Specific Attributes for Substructure

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	NCHRP 12-82 Degree of Severity	INDOT Degree of Severity	Max Score
C.5	Maintenance Cycle	No maintenance			Washing / Sealing	H	M	15
C.11	Presence of Repaired Areas	Significant amount of repaired areas	Moderate amount of repaired areas	Minor amount of repaired areas	No repaired areas	M	L	10
C.13	Efflorescence/ Staining	Efflorescence with rust staining	Moderate Efflor.	Minor Efflor. without rust staining	No Efflor.	H	M	15
D.11	Concrete Cover	<1.5"	1.5" - 2.5"	2.5"+		H	M	15

Consequence factor was determined for the damage mode of pier corrosion by the RAP. The consensus was that pier corrosion had a Moderate consequence, as shown in Table 4.14. Site specific conditions such as feature under, feature carried, and traffic volume could change the consequence to Low or High. For example, a substructure spall from a bridge spanning a non-navigable waterway on short piers would have a lower consequence than a flyover bridge spanning an interstate. The former is not a safety concern for the traveling public, while a spall from the latter could fall onto the interstate and pose a safety hazard. It was not expected that pier corrosion would cause bridge collapse. Ultimately, the RAP consensus was that substructure corrosion could have a consequence factor of Low, Moderate, or High dependent upon site specific conditions at each bridge.

Table 4.14: RAP Results for Pier Corrosion Consequence Factor

Pier Corrosion					
		Low	**Moderate**	**High**	**Severe**
1	Participant 1	-	50	50	-
2	Participant 2	90	10	-	-
3	Participant 3	50	30	20	-
4	Participant 4	-	-	80	20
5	Participant 5	-	70	30	-
6	Participant 6	-	60	40	-
7	Participant 7	10	30	50	10
8	Participant 8	10	40	40	10
9	Participant 9	-	100	-	-
10	Participant 10	-	50	50	-
11	Participant 11	60	30	10	-
		0.20	**0.43**	**0.34**	**0.04**
The Consequence of this Damage Mode is Moderate.					

4.7. <u>Indiana RAP Summary</u>

The Risk Assessment Panel (RAP) meeting was a consensus-based expert elicitation approach utilized to develop an Indiana specific risk-based inspection approach. The Indiana RAP was composed of INDOT personnel and industry consultants. Using an expert elicitation process, RAP members identified credible damage modes for concrete decks, steel superstructures, and prestressed concrete superstructures. Concrete superstructures and substructures were not specifically addressed in this meeting. Relevant design, condition, loading and screening attributes were identified and ranked for importance. This ranking (high, medium, or low) was used to establish a preliminary scoring method to determine occurrence factor. Consequence factors for the determined damage modes were also identified by the Indiana RAP. After the meeting, the information was analyzed and organized into scoring models. Back-casting, which involved monitoring deterioration progression through historical data and comparing the results with the risk approach, was used to evaluate the safety and effectiveness of the RAP specific risk model.

CHAPTER 5. BACK-CASTING RESULTS FOR INDIANA

To evaluate that the risk-based procedure could establish a safe and effective inspection interval, a process called back-casting was performed. Back-casting involved monitoring deterioration progression through historical data, and then comparing the results with the risk approach. Two trials for back-casting in Indiana were performed. The first trial used the general criteria set out by the NCHRP 12-82 study. The second trial occurred after the Indiana RAP meeting, and used the criteria developed during the Indiana RAP meeting. The overall inspection intervals determined from both trials for all bridges were identical. Therefore, only the back-casting analysis with the Indiana specific criteria will be described. An overview of the procedure and analysis will be presented, followed by three back-casting examples. Overall, there were no cases where a bridge deteriorated to a serious condition during the inspection intervals as a result of the proposed procedures.

5.1. Back-Casting Overview and Source of Data

After completing the expert elicitation process, the risk criteria, damage modes, and attributes from the Indiana RAP meeting were evaluated and compiled. Using the results from the RAP, the methodology was then revised to be best aligned with the bridge inventory in Indiana. Validation of the revised procedures was performed through a process called back-casting. Data for the back-casting process was acquired from the Indiana Department of Transportation's historical bridge inspection record databases.

5.1.1. Back-Casting Concept

The back-casting process involved using historical inspection records to monitor the deterioration of a sampling of bridges, and then comparing the deterioration progression with the inspection interval predicted by the proposed risk method. For example, using inspection data from 1980 for a given bridge, the risk procedures herein were applied. If the inspection interval determined was six years, the next back-casting assessment would occur in 1986. Then, the historical inspection reports (e.g., from 1986) would be used as a "check" that no major deficiencies or deterioration occurred or were not accounted for by the RBI procedure during the extended six year interval. The process would then be repeated for the life of the bridge. This approach would effectively reveal the adequacy or shortcomings of the approach, allow for calibrated adjustments, and increase confidence in the risk procedure overall.

5.1.2. Source of Data

Obtaining historical inspection data was a critical component of accurate back-casting. Historical inspection reports for over sixty bridges were acquired, and after compiling the inspection data, thirty-six bridges were determined to have sufficient historical information to accurately complete a back-casting assessment. The data was stored in three different formats: microfilm, the Electronic Records Management Software (ERMS) database, and the Bridge Inspection Application System (BIAS) database.

5.1.2.1. Microfilm

In the 1990s, Indiana archived old bridge inspection reports using microfilm. To obtain inspection data from 1980 – 2000, selected bridges were found in the microfilm roll index. Once the proper roll and inspection report were located, individual reports were printed to pdf. Early inspection reports typically consisted of four pages and had sections for general information, deck, superstructure, substructure, channel & channel protection, culvert & retaining wall, estimated remaining life, approach alignment, rated loading, appraisal, and proposed improvements. An area for comments was also present.

Back-casting assessments in this time frame were made based upon the provided comments and condition ratings.

As computers became more commonly used in bridge inspection database management, the inspection report increased in detail. A typical report from 1992-2000 consisted of ten pages, with the sections mentioned above plus the addition of paint condition, scour/erosion, critical features, district priority, structural details, collision damage, actions taken, and roadway management data. Individual components of the bridge such as bolts, splice plates, welds, and hanger bars were given a condition rating. Pictures, if taken, were not included in the archived reports. Back-casting assessments in this time frame utilized this added data.

5.1.2.2. ERMS Database

From around 2000-2008, the computerized ERMS database was used to collect inspection data for Indiana bridges. These reports included individual component condition rating assessments, as well as a large section for comments. They were typically eight pages in length. Photos were taken with these reports; however, they were not included in the electronic file. Therefore, back-casting assessments in this period relied upon inspector comments and condition ratings.

5.1.2.3. BIAS Database

The current database used to collect inspection data is BIAS. Detailed inspection reports are over 50 pages long and include multiple pages of pictures. Each major component has a section for inspector comments. Commentary, condition ratings, and visual evidence from pictures were utilized for back-casting.

5.2. Indiana Bridge Inventory

Thirty-six bridges were selected for use in the back-casting study. Bridges were randomly selected based upon superstructure type, geographical region, and current condition rating to create a representative sample. The distribution of superstructure types was nearly an even split between steel, reinforced concrete, and prestressed concrete. Table 5.1 shows the breakdown.

Table 5.1: Distribution by Superstructure Type for Back-Casting

Superstructure Type	Number of Bridges
Reinforced Concrete	13
Steel	12
Prestressed Concrete	11

Each geographical district in Indiana was represented. However, incomplete historical inspection data from certain districts made it difficult to have an equal distribution between the districts. Locations of the bridges used in back-casting can be seen in Figure 5.1, and the distribution between districts is reported in Table 5.2. Bridges that had progressed to condition rating CR 3 or CR 4 were used to ensure that an adequate bridge history existed, and to ensure that the risk procedure captured the deterioration.

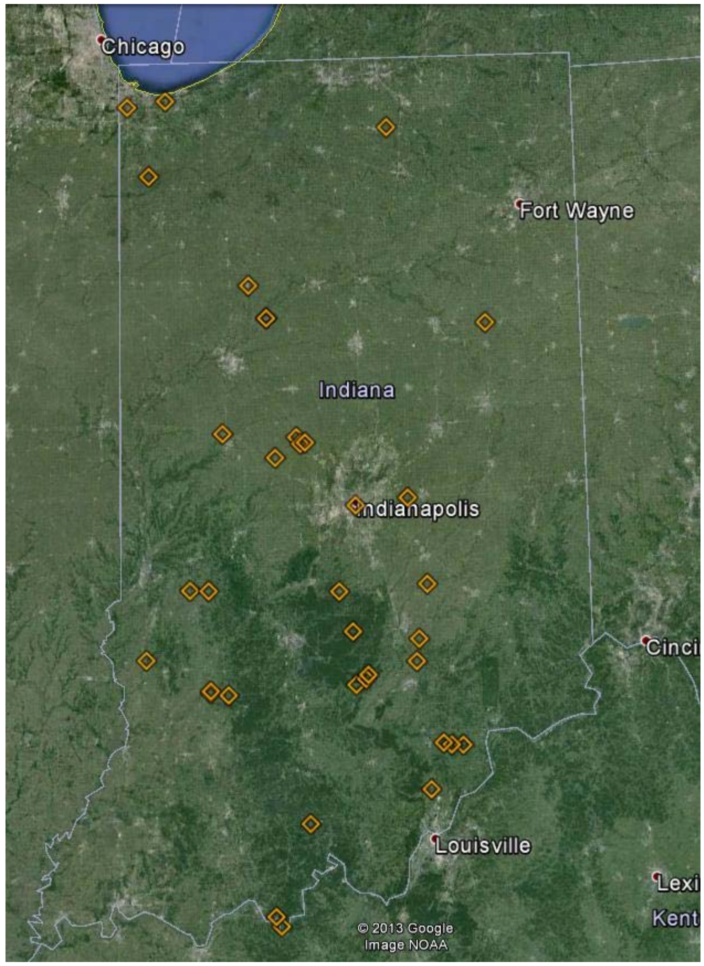

Figure 5.1: Geographical Distribution of Indiana Bridges for Back-Casting Study

Table 5.2: Distribution by District for Back-Casting Study

District	Number of Bridges
Crawfordsville	8
Fort Wayne	2
Greenfield	3
LaPorte	5
Seymour	11
Vincennes	7

5.3. Inspection Intervals

Maximum inspection intervals were determined based on a risk matrix. Each damage mode was considered individually, with a distinct occurrence factor and consequence factor. Entering the risk matrix with the occurrence factor and consequence factor determined the inspection interval. The damage mode with the shortest inspection interval was the controlling factor for that bridge. In some cases, the controlling factor changed as the bridge aged, e.g., initially, deck cracking may have controlled and then as the bridge deteriorated, superstructure corrosion may have controlled. A summary on how the occurrence factor, consequence factor, and inspection intervals were determined for the back-casting procedure follows.

5.3.1. Determining the Occurrence Factor

To determine the occurrence factor, credible damage modes for each bridge component were first established. Then, attributes were associated with each damage mode, and given a point value based upon their relative importance as established by expert elicitation. The greater the number of points a bridge received, the higher the occurrence factor.

An application within Microsoft Excel was created for this research to facilitate conducting multiple risk assessments during the back-casting process. Damage modes and attributes from the RAP meeting were selected and organized along the left side of the spreadsheet. On the right side of the spreadsheet, point values for the various attributes were recorded, and once completely entered, the occurrence factor was calculated automatically. The spreadsheet enabled the first assessment for each bridge to be performed in 30 minutes, and subsequent assessments to be performed in 15 minutes. The time savings comes from previously coded and unchanging design attributes such as material type, concrete cover, and environment of the bridge. In the future, more advanced software programs could be developed, and possibly attached to the current database as a tool to enable efficient bridge risk assessments.

A portion of the excel spreadsheet utilized in this study is shown in Figure 5.2. The figure displays the damage mode of corrosion between beam ends for a concrete girder, and some of the attributes and point values associated with it. Assessments from 2008 and 2012 are shown. For this bridge, it can be seen that an increase in deterioration in the form of spalling and delaminations resulted in an increase in the points recorded. In this specific case, the occurrence factor for the damage mode of corrosion between beam ends was increased from Moderate in 2008 to High in 2012 due to the increase in spalling and delamination present. However, any increase in received points does not automatically correspond to an increase in occurrence factor.

Corrosion Between Beam Ends -- Concrete Girder			Score	Score
Attribute		Points	2008	2012
C.1 Current Condition Rating				
	Current rating is 5 or below	20	20	20
	Current rating is 6	5		
	Current rating is 7+	0		
C.6 Previously Impacted				
	Bridge has been previously impacted	20		
	Bridge has not been previously impacted	0	0	0
C.8 Corrosion Induced Cracking				
	Significant corrosion induced cracking	20		
	Moderate corrosion induced cracking	10		10
	Minor corrosion induced cracking	5	5	
	No corrosion induced cracking	0		
C.9 General Cracking				
	Widespread or severe cracking	15	15	15
	Moderate cracking present	10		
	Minor or no cracking	0		
C.10 Delaminations				
	Unknown	20		
	Significant (>20% by area) delamination	20		20
	Moderate (5-20% by area) delamination	10	10	
	Minor, localized (<5% by area)	5		
	No delamination present	0		
C.11 Presence of Repaired Areas				
	Significant amount of repaired areas	15		
	Moderate amount of repaired areas	10		
	Minor amount of repaired areas	5		
	No repaired areas	0	0	0
C.12 Presence of Spalling				
	Significant spalling (>10% by area with exposed rebar or strands)	20		20
	Moderate spalling (greater than 1 inch deep or 6 inches diameter exposed reinforcement)	15	15	

Figure 5.2: Example Screen from a Software Application Demonstrating a Damage Mode and Attributes for the Risk Assessment

5.3.2. Determining the Consequence Factor

Expert elicitation was used to determine the consequence factor for all of the damage modes. Immediate consequence looked at the safety aspect of a scenario, while the short-term consequence reflected the serviceability aspect. Both immediate and short-term consequence were considered when making an assessment. In-depth information about the immediate and short-term consequence can be found in Chapter 3.

The experts at the Indiana RAP agreed that the consequence factors depended primarily on the bridge's traffic volume, feature carried, feature under, presence of stay-in-place forms, redundancy, composite action, and load carrying capacity. In general, a higher traffic volume would indicate a higher overall consequence because of the short-term consequence. For example, closing a shoulder or lane on an interstate would create a greater impact than closing a shoulder or lane on a rural highway.

Additionally, from a consequence perspective, bridges over roadways were treated differently than bridges over non-navigable waterways. Falling debris from bridges over roadways could create a safety issue to the people beneath the bridge, while falling debris from bridges over non-navigable waterways would not be expected to present a safety hazard to the traveling public. Therefore, the Indiana RAP decided that bridges over roadways and multi-use paths would automatically have a High consequence for components of the bridge that could fall onto the public below. Non-composite bridges are also included in this category. Bridges over non-navigable waterway and railroads were considered to have a Low or Moderate consequence factor based upon traffic volume and feature carried.

Redundancy was another consideration in the consequence determination. While typical redundant highway bridges are the emphasis for the proposed risk procedure, the risk assessment has potential application for fracture critical bridges. Fracture critical bridges receive the highest consequence factor, Severe, because the immediate consequence is assumed to be structural collapse. In general, bridges that meet the AASHTO criteria for superstructure redundancy will have either a High or Moderate consequence factor for the superstructure. Redundant bridges are expected to have adequate structural capacity after the loss of a load bearing member, and if documented experience or engineering analysis exists as supporting evidence, the immediate consequence can be reduced to Moderate.

Ultimately, the consequence factor is a combination of the immediate and short-term consequence. Immediate consequence encompasses structural and public safety, while short-term consequence includes serviceability and effects to the traveling public. Maintenance and repair costs were not directly considered. Determining the consequence factor for back-casting followed the guidance set out by expert elicitation and the Indiana RAP.

5.3.3. Determining the Inspection Interval

The inspection intervals were determined through the use of a risk matrix. Figure 5.3 show the typical highway bridge risk matrix used for the back-casting analysis. Consequence factor is along the x-axis, and the occurrence factor is along the y-axis. The figure also illustrates the applicable inspection interval based upon the occurrence factor and the consequence factor.

Occurrence Factor

High	**48** months	**24** months	**24** months	**12** months
Moderate	**48** months	**48** months	**24** months	**24** months
Low	**72** months	**72** months	**48** months	**24** months
Remote	**96** months	**72** months	**48** months	**48** months
	Low	Moderate	High	Severe

Consequence Factor

Figure 5.3: Risk Matrix for Indiana Back-Casting

5.4. <u>Back-Casting Examples</u>

To further clarify the back-casting process, three examples are presented. The first example is a 56 year old steel superstructure bridge located in a southern district. Next is a 38 year old concrete superstructure bridge located in a northern district, and the third is a 50 year old prestressed bridge also located in a northern district.

5.4.1. Bridge Number: I65-14-04218B

Bridge number I65-14-04218B is a four-span continuous steel beam structure with a concrete cast-in-place deck located in Clark County, Indiana. Built in 1958, it carries a two lane road with an average daily traffic (ADT) of 400 vehicles per day, and spans across I-65 which has an approximate ADT of 19,500 vehicles per day. Rivets and bolts are both present in the superstructure. With a clearance of 14’-07”, the bridge also has a history of impact damage. The substructure is concrete. Condition ratings in 2012 were CR 6 for the deck, CR 3 for the superstructure, and CR 7 for the substructure.

Figure 5.4: Views of bridge I65-14-0218B

5.4.1.1. Occurrence Factor

Two major considerations for the occurrence factor were fatigue and fracture. The bridge was fabricated prior to the industry-wide development of the fracture control plan (FCP) and was designed prior to the inclusion of fatigue provisions in the AASHTO (then AASHO) specifications. Hence, bridges from this period are expected to be more susceptible to fatigue and fracture. Major attributes for fatigue were age, connection type, fatigue detail category, and average daily truck traffic (ADTT). This bridge received points in all categories, and had a Moderate occurrence factor for the fatigue damage mode. Fracture was the second major consideration, and had a Moderate and then a High occurrence factor; previous impact history, vertical clearance, and year of construction were the major considerations. When cracking in the girder from an impact was discovered, the increase in points awarded resulted in a High occurrence factor. Overall, fatigue and then fracture controlled the likelihood of failure for this bridge.

5.4.1.2. Consequence Factor

The consequence scenario considered for the deck was spalling. From this damage mode, the immediate consequence factor was Moderate. Metal forms on the bridge prevent spalled concrete from the bottom of the deck from reaching the roadway.

Additionally, the top of the deck was considered to present a minimal safety concern to the traveling public based upon the low ADT. Short-term and serviceability concerns were also rated as Moderate. A lane closure or speed reduction on the bridge would moderately impact traffic. Therefore, considering both immediate and short-term effects, the consequence factor for the deck was determined to be Moderate.

The consequence scenario considered for the superstructure was the loss of a primary load bearing member. The immediate consequence was considered to be High. The bridge is a multi-girder redundant bridge by AASHTO standards, so the structural capacity was expected to remain adequate. If structural analysis or documented experience existed for this bridge, it may be possible to reduce the immediate consequence factor to Moderate. It was not expected that the structure would collapse. Short-term consequence considered the effect of closing a lane on the interstate for bridge repair. Because the interstate has a high traffic volume, the traveling public would be greatly impacted. Based on these considerations, the consequence factor for the superstructure was determined to be High.

Corrosion was the consequence scenario considered for the substructure. The immediate consequence factor from spalling concrete was considered to be Moderate. Falling debris from the structure presents a minimal safety concern to the interstate due to the proximity. Short-term consequence was also considered to be Moderate due to the low traffic volume on the bridge. Load posting or speed reduction as a result of corrosion would moderately impact traffic. Therefore, the substructure consequence was Moderate.

5.4.1.3. Interval

Back-casting ratings for this bridge began in 1982. The inspection interval was consistently 24 months based upon an occurrence factor of Moderate combined with a consequence factor of High for the superstructure. The occurrence factor was Moderate for the fatigue damage mode because the bridge was fabricated prior 1975, was connected with rivets, had Category D fatigue details, and had an unknown remaining fatigue life. The average daily truck traffic (ADTT) on the bridge was less than one-hundred vehicles per day, so the traffic attribute did not contribute to the score. The fatigue damage mode controlled the inspection interval from 1982-1992. Then, in 1993, inspectors noticed cracking in the steel girders as a result of previous impacts, and fracture became the controlling damage mode for the occurrence factor. The impact damage can be seen in the right picture of Figure 5.4. The attributes that contributed to the High occurrence factor were: the presence of cracks in the girders, a fabrication date prior to 1975, a vertical clearance less than 15 feet, material that was not high performance steel, previous impacts, and an unknown fatigue life. Again, ADTT was an attribute that related to fracture, but did not score because of the low traffic volume on the bridge. The Moderate occurrence factor combined with the High consequence factor resulted in a 24 month inspection interval. The dates of the historical inspection reports and their corresponding inspection intervals can be seen in Figure 5.3. Even with different controlling damage modes, the risk method recommends an inspection interval of 24 months.

Table 5.3: Inspection intervals for bridge I65-14-04218B

Year	1982	1984	1986	1988	1993	1994	1996	1998	2000
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2002	2003	2005	2007	2009	2011	2012		
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months		

As an example, the risk matrix for the assessment from 2012 is illustrated in Figure 5.5. Overall, the inspection interval is 24 months based upon the controlling damage mode of fracture with a High occurrence factor and a High consequence factor. The damage modes of fatigue and superstructure corrosion have a Moderate occurrence factor and a High consequence factor, which is a 24 month inspection interval. Deck and substructure corrosion both have a Low occurrence factor and a Moderate consequence factor, which would indicate a 72 month inspection interval. However, the deck and substructure damage modes do not control the inspection interval for this bridge. Therefore, the recommended 2012 inspection interval for this bridge based upon the risk methodology is 24 months.

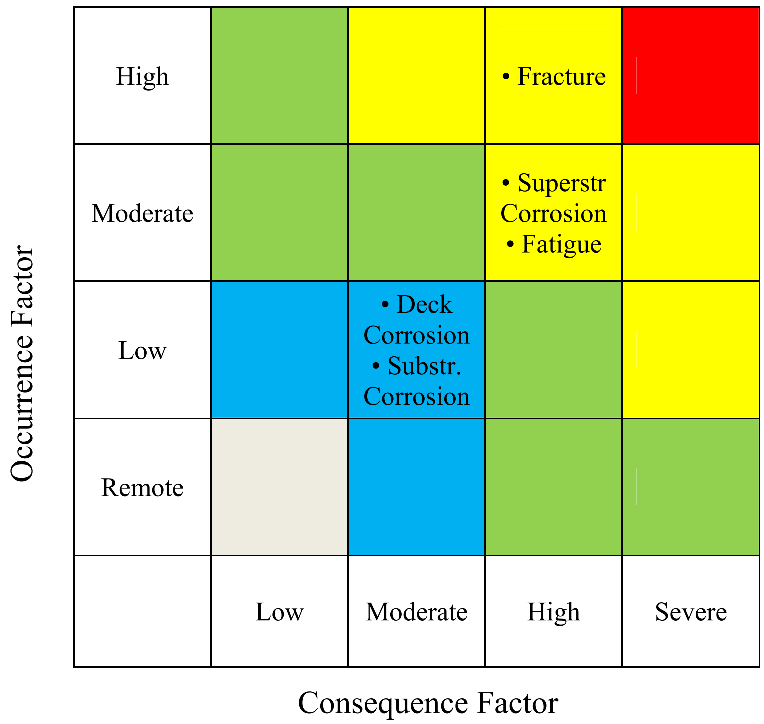

Figure 5.5: Risk Matrix for Bridge I65-14-04218B

5.4.2. Bridge Number: 45-28-03529

Bridge number 45-28-03529 is a three-span continuous reinforced concrete slab structure with a concrete cast-in-place deck located in Greene County, Indiana. Built in 1946, it carries a two lane road with an ADT of 2600 vehicles per day, and intersects Doan's Creek. The substructure is concrete and scour is not a concern. Condition ratings as of 2012 are CR 4 for the deck, CR 4 for the superstructure, and CR 4 for the substructure.

Figure 5.6: View of bridge 45-28-03529

5.4.2.1. Occurrence Factor

The progression of corrosion was the major consideration for the occurrence factor. Main attributes for corrosion were age, concrete mix design, concrete cover, reinforcement type, exposure environment, presence of cracks, and current condition. As the bridge corroded over time, the likelihood of failure increased from Low to High. The methodology tracked the deterioration and indicated increasing risk of failure with increasing corrosion.

5.4.2.2. Consequence Factor

The consequence scenario considered for the deck was spalling. From this damage mode, the immediate consequence factor was Low. The bridge does not have metal forms for the deck; however, falling debris from the bottom of the deck will drop into a non-navigable waterway. The safety of the public is unaffected. Additionally, the top of the deck was considered to present a minimal safety concern to the traveling public based upon the low ADT. Short-term and serviceability concerns were also rated as Low. A lane closure or speed reduction on the bridge would have a minor impact on traffic. Therefore, considering both immediate and short-term effects, the consequence factor for the deck was determined to be Low.

The consequence scenario considered for the superstructure was the loss of a primary load bearing member. The immediate consequence was considered to be Moderate. The bridge is a multi-girder redundant bridge by AASHTO standards, so the structural capacity was expected to remain adequate. It was not expected that the structure would collapse. Additionally, like the deck, falling spalled concrete would not affect the safety of the public because the feature intersected was a non-navigable waterway. Short-term consequence considered the effect of closing a lane on the road for bridge repair. Because of the low traffic volume, the traveling public would be minimally impacted. Based on these considerations, the consequence factor for the superstructure was determined to be Moderate.

Corrosion was the consequence scenario considered for the substructure. The immediate consequence factor from spalled concrete was considered to be Low. Falling debris from the structure does not present a safety concern to the traveling public because the bridge spans a non-navigable waterway. Short-term consequence was also considered to be Low due to the traffic volume on the bridge. Minor serviceability concerns may require maintenance. Therefore, the substructure consequence was Low.

5.4.2.3. Interval

Back-casting ratings for the bridge began in 1980 and the inspection interval determined was 72 months. Corrosion was the controlling damage mode with an occurrence factor of Low. This was based upon design and loading attributes such as age, ADTT, and material type. Condition attributes included delaminations, general cracking, presences of repaired areas, and presence of spalling. Because the bridge was in good condition, there was little deterioration and the condition attributes did not add to the occurrence factor score. In 1986, the inspection interval remained at 72 months; however, the bridge was beginning to show small signs of corrosion, especially in the substructure. The assessment in 1990 revealed continuing substructure corrosion which caused an increase in the occurrence factor to Moderate and a reduction of the inspection interval to 48 months. Corrosion in the deck and superstructure progressed to a Moderate occurrence factor in 1997. The slowly progressing corrosion in all three bridge components was the controlling damage mode for the next 20 years. During this time, the bridge did decrease in NBI condition rating as shown in Figure 5.7. However, the risk method does not attempt to predict or track the change in condition ratings, and looks at the likelihood of failure and the consequence of failure. Therefore, changes in condition rating do not always coincide with changes in inspection intervals, though the relationship can be complex. In 2010, the inspection interval dropped to 24 months based upon the superstructure corrosion damage mode. There was significant spalling, delamination, cracking, and efflorescence exhibited by the reinforced concrete girders. In addition, the condition rating of the deck and superstructure dropped to CR 4. This is a screening condition in the methodology, and automatically recommends a 24 month inspection interval for the bridge because there is a decrease in the reliability of the bridge component. Overall, as the bridge deteriorated, the inspection interval decreased.

Table 5.4: Inspection intervals for bridge 45-28-03529

Year	1980	1982	1985	1987	1990	1993	1997	1999	2001
Inspection Interval	72 months	72 months	72 months	72 months	48 months	48 months	48 months	48 months	48 months
Year	2002	2004	2008	2010	2012				
Inspection Interval	48 months	48 months	48 months	24 months	24 months				

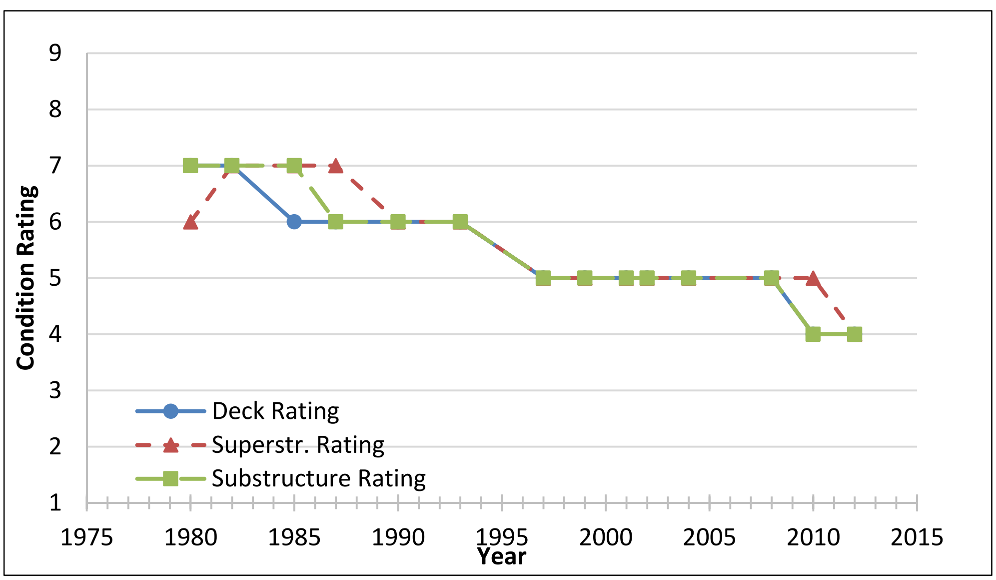

Figure 5.7: NBI condition rating for bridge 45-28-03529 from 1980-2012

As an example, the risk matrix for the assessment from 2012 can be seen in Figure 5.8. Currently, the maximum inspection interval is 24 months based upon the controlling damage mode of superstructure corrosion with a High occurrence factor and a Moderate consequence factor. Substructure corrosion has a Moderate occurrence factor and a Low consequence factor resulting in a 48 month inspection interval, while deck corrosion has a Low occurrence factor and a Low consequence factor, resulting in a 72 month inspection interval. The damage modes of shear and flexural cracking of the superstructure were non-issues for this bridge, with a Remote occurrence factor indicating they were not likely to affect the reliability of the bridge as a whole. Therefore, the recommended 2012 inspection interval for this bridge in 24 months.

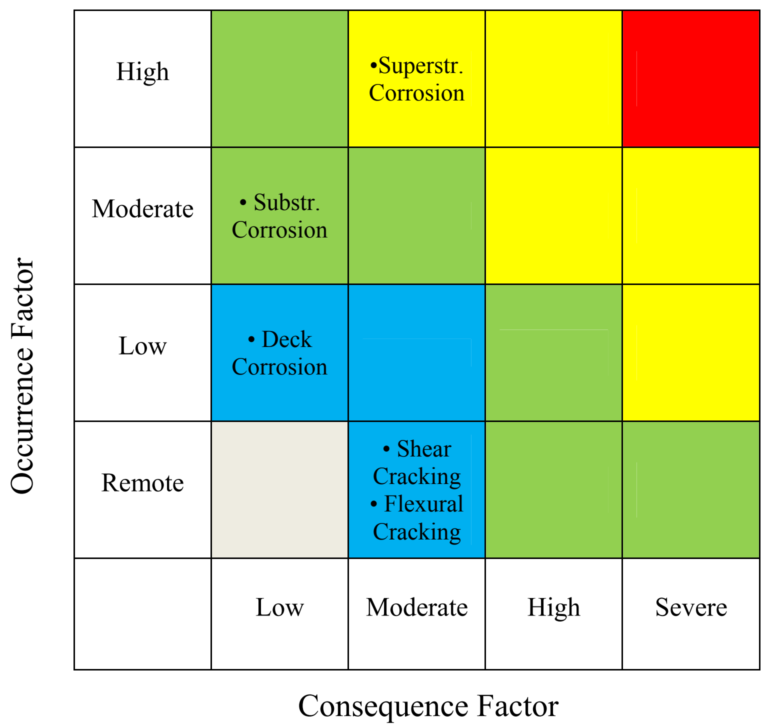

Figure 5.8: Risk Matrix for Bridge 45-28-03529

5.4.3. Bridge Number: 55-45-06258B

Bridge number 55-45-06258B is a three-span prestressed concrete box beam structure with a concrete cast-in-place deck located in Lake County, Indiana. Built in 1964, it carries a two lane road with an ADT of 1700 vehicles per day, and intersects

Singleton Ditch. The substructure is concrete and scour is not a concern. Condition ratings as of 2012 are CR 3 for the deck, CR 4 for the superstructure, and CR 4 for the substructure.

Figure 5.9: View of bridge 55-45-06258B

5.4.3.1. Occurrence Factor

Corrosion in all components of the bridge was the primary consideration for the occurrence factor. Main attributes for corrosion were age, concrete mix design, concrete cover, reinforcement type, exposure environment, presence of cracks, and current condition. Signs of deterioration for the deck and substructure were cracking, spalling, and efflorescence. As time progressed, the superstructure exhibited corrosion in the forms of cracking and rust staining. Overall, the corrosion damage mode had the highest occurrence factor.

5.4.3.2. Consequence Factor

The consequence scenario considered for the deck was spalling. The immediate consequence factor was Low based upon feature under—a non-navigable waterway. Falling debris from the bottom of the deck is not a safety concern for the public. The top of the deck was considered to present a minimal safety concern to the traveling public based upon the low ADT. Short-term consequence was also assessed as Low. A lane closure or speed reduction on the bridge would have a minor impact on traffic.

Therefore, considering both immediate and short-term effects, the consequence factor for the deck was determined to be Low.

The consequence scenario considered for the superstructure was the loss of a primary load bearing member, and the immediate consequence was considered to be Moderate. The bridge is a multi-girder redundant bridge by AASHTO standards, and the structural capacity was expected to remain adequate. Similar bridges that have lost a load bearing member have remained standing. Thus, it was not expected that the structure would collapse. Additionally, like the deck and substructure, falling spalled concrete would land in the non-navigable waterway or unused right-of-way land. Short-term consequence considered the effect of closing a lane on the road for bridge repair. Because of the low traffic volume, the traveling public would be minimally impacted. Based on these considerations, the consequence factor for the superstructure was determined to be Moderate.

Corrosion was the consequence scenario considered for the substructure, and the immediate consequence factor from spalling concrete was considered to be Low. Falling debris from the structure would fall into a non-navigable waterway and does not present safety concern to the traveling public. Short-term consequence was also considered to be Low based upon the low traffic volume on the bridge. In the event of a lane or shoulder closure, little or no impact to the traveling public is expected. Therefore, the substructure consequence was Low.

5.4.3.3. Interval

Back-casting ratings for the bridge began in 1983, and the inspection interval determined was 72 months based upon the damage mode of deck corrosion. The Low occurrence factor was influenced by the moderate exposure environment, indications of ineffective drainage, the fabrication date, and the non-high performance concrete mix. The interval remained at 72 months until 2001, when it decreased to 48 months based upon the damage modes of deck and substructure corrosion. At that time, the bridge exhibited moderate corrosion induced cracking, moderate spalling, moderate delaminations, and multiple repaired areas. The bridge had multiple potholes in the deck, and sections of spalling on the substructure. The inspection interval remained at 48 months through 2009, when the condition rating dropped to CR 4, effectively screening out the bridge and assigning a 24 month inspection interval for reliability reasons. In 2010, the damage mode of superstructure corrosion had an occurrence factor of High, which combined with the consequence factor of Moderate resulted in a 24 month inspection interval. Overall, as the bridge deteriorated, the inspection interval decreased.

Table 5.5: Inspection intervals for bridge 55-45-06258B

Year	1983	1987	1989	1991	1993	1995	1997	1999	2001
Inspection Interval	72 months	72 months	72 months	72 months	72 months	72 months	72 months	72 months	48 months
Year	2007	2009	2010	2012					
Inspection Interval	48 months	24 months	24 months	24 months					

The risk matrix for the assessment from 2012 can be seen in Figure 5.10. Currently, the maximum inspection interval is 24 months based upon the controlling damage mode of superstructure corrosion that has a High occurrence factor and a Moderate consequence factor. Deck and substructure corrosion both have a Moderate occurrence factor combined with a Low consequence factor for a 48 month inspection interval. Strand fracture in the superstructure has a Low occurrence factor and a Moderate consequence factor, for a 72 month inspection interval. Flexural and shear cracking were not concerns for this bridge, with a Remote occurrence factor and a Moderate consequence factor. Overall, the damage mode of superstructure corrosion control the interval, and the risk procedure recommends a 24 month inspection interval.

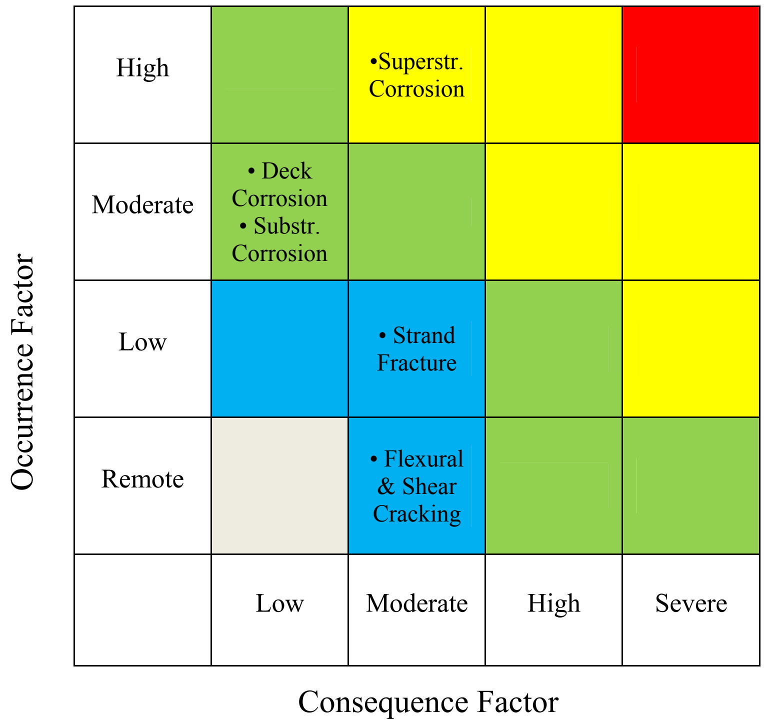

Figure 5.10: Risk Matrix for Bridge 55-45-06258B

5.5. Back-Casting Summary

Back-casting evaluated the safety and effectiveness of the risk procedure for determining suitable inspection intervals up to 72 months for typical highway bridges. A representative sample of thirty-six Indiana bridges was considered, and there were no cases where a serious progression of damage would have been missed as a result of the proposed methodology. Fourteen of the thirty-six bridges had an inspection interval of 72 months at some point during the back-casting process and twenty-one bridges had a 48 month interval at some point during the process. Bridges in poor condition were assigned inspection intervals of 24 months. In addition, no unexpected or sudden changes to the NBI condition rating were noted during the risk-based inspection interval. In some cases, bridges in good condition ratings had short inspection intervals assigned based upon risk factors not revealed through condition rating alone. Based upon the successful back-casting evaluation performed, risk-based inspection intervals up to 72 months appear to be safe, effective, and implementable for the state of Indiana using the criteria developed during the RAP meeting.

CHAPTER 6. FAMILIES OF BRIDGES

Families of bridges were created to recognize the similarity of design, condition, and loading attributes in the risk process. Bridges in a family have similar damage modes and are expected to deteriorate in the same fashion at nearly the same rate. Multiple families for the Indiana inventory are proposed. An evaluation of the Indiana inventory was also conducted to determine the inspection intervals for the proposed families. The concept of bridge families, surrogate data, proposed families of bridges for the Indiana inventory, applications for RBI in Indiana, and an implementation strategy are explored.

6.1. Concept and Process

A family of bridges is a group of bridges with similar design, condition, and loading attributes that are expected to deteriorate with the same damage modes at approximately the same rate. Families can be determined based upon a number of characteristics. For example, bridges built before the implementation of the fracture control plan have different design parameters than bridges built after and could be grouped accordingly. Bridges can also be grouped based upon superstructure type, geographical location, environment, date built, maximum span length, or any combination of attributes. The key is to identify bridges that are expected to behave in nearly identical manners during the inspection interval.

Creating families of bridges can increase the efficiency of determining inspection intervals. Bridges with similar attributes in similar environments are likely to have the same occurrence factor. Therefore, the occurrence factor for a family can be calculated once per cycle and assigned to the entire family. If desired, the families could also include a criteria for consequence factor. Then, the inspection interval would be known for an entire family based upon the determined occurrence factor and consequence factor. For example, perhaps the RAP decided that prestressed box beam superstructures built after 2000 have similar attributes and can be considered a family. Then, if it is assumed the occurrence factor is Low, bridges with a Moderate consequence, such as those over non-navigable waterways or carrying a rural highway, would have an inspection interval of 72 months. As the occurrence factor changes based upon the inspection data, the inspection interval would also change. The inspection report from every bridge would need to be reviewed; however, each bridge would not need to have an individual risk assessment, unless the inspection report revealed unusual damage or rapid deterioration.

Determining families of bridges occurs at the owner level during the RAP meeting. Familiarity with the bridge inventory and typical deterioration patterns is essential to effectively create families of bridges. RAP members determine critical attributes that can be used to identify bridges with similar deterioration expectations, and group bridges. Sometimes, not all information is known about a bridge and surrogate data, or data that can be used to infer a required piece of information, is used to supplement the existing data. Families can range in size; however, creating small families may not be as efficient as creating larger families for rating purposes. Ultimately, the RAP should decide upon families of bridges that are feasible and practical for their inventory.

6.2. Surrogate Data

To improve the efficiency of risk analysis for families of bridges, surrogates for the attributes can be considered. Surrogate data is specific data that can be used to infer or determine a required piece of information for the risk assessment. For example, any bridge designed and built after the implementation of the AASHTO Fracture Control Plan in 1974 can be inferred to have steel that meets certain toughness requirements and that meets modern fatigue provisions. This information was inferred from the date of construction only and did not require a plan review.

Year of construction is a useful surrogate. From year of construction, the fatigue and fracture resistance of the bridge can be inferred based upon whether the bridge was designed before or after the implementation of the AASHTO Fracture Control Plan. Bridges designed after the plan was enacted are expected to have a higher resistance to fatigue and fracture events. Variation in materials and fracture toughness exist in bridges built before 1975 that may decrease reliability. Ultimately, this results in a higher occurrence factor based upon the scoring process. The year of construction is also a surrogate for concrete cover. In 1970, the recommendation for clear concrete cover was two inches. Greater uncertainty for depth of cover exists in bridges built prior to 1970. The 2002 AASHTO standards require a minimum of 2.5 inches of concrete cover for uncoated reinforcing steel. Therefore, depending on when the bridge was built, the concrete cover can be inferred.

Current condition rating can also be used as surrogate for condition attributes. Based upon the subjective condition rating statements found in the NBIS, the current level of bridge component deterioration can be inferred. For example, bridges in condition rating CR 9 are expected to be in virtually new condition. Bridges in condition rating CR 8 have no noted problems, and bridges in CR 7 have some minor problems. The description for condition rating CR 6 includes minor deterioration. Therefore, for the risk assessment, it can be inferred that bridges in CR 7 or better have no deterioration or have very minor deterioration and are awarded the lowest level of points for condition attributes.

As a clarification, inspection intervals are not assigned based upon the current condition rating of the bridge and do not always change when the condition rating changes. For example, deciding to assign all steel bridges in condition rating CR 7 on a longer interval than CR 6 does not take into account the design and loading attributes as well as the likelihood of failure and is therefore not recommended. The risk process is based upon expert elicitation and engineering rationale that considers the condition of the elements, design characteristics, loading characteristics, the likelihood of damage, and the consequences of damage. While current condition rating can be used as a surrogate, it is not the only influential factor in determining inspection interval.

6.3. Proposed Families

Many bridges in the Indiana inventory have similar design characteristics. Based upon key attributes and the scoring method developed by the RAP, families of bridges can be developed. A family includes bridges with similar attributes that are expected to deteriorate in a similar fashion at a similar rate. The following families were identified for Indiana.

6.3.1. High Rated

Bridges currently in good condition e.g., condition rating CR 9, CR 8, or CR 7, can be inferred to have no deterioration or very minor deterioration. The condition attributes are therefore awarded the lowest level of points. A family of bridges that have high condition ratings can be useful to group together because a low likelihood of failure and a Low occurrence factor is expected. Bridges with the following characteristics are considered a part of this family:

- Deck condition rating CR 7 or better
- Superstructure condition rating CR 7 or better
- Substructure condition rating CR 7 or better
- Built in 1975 or after
- Not fracture critical
- Not scour critical
- No impact damage

Bridges in condition rating CR 9 are expected to be in virtually new condition, while bridges in condition rating CR 8 have no noted problems, and bridges in CR 7 have some minor problems. The description for condition rating CR 6 includes minor deterioration. For the risk assessment, it can be inferred that bridges that have all components in CR 7 or better have no deterioration or very minor deterioration and are therefore awarded the lowest level of points for condition attributes. As a result, most bridges in CR 9, CR 8, or CR 7 have a Low occurrence factor. Design and loading attributes must also be considered; therefore not all bridges in good condition rating have an extended inspection interval. The key deciding attributes for whether the bridge will fall into the High Rated Family of bridges are described below.

Year built is the first key attribute. Over the years, design specifications have adapted and improved with the growing civil engineering knowledge base. The modern fatigue design provisions were incorporated in 1975; thus, bridges designed prior to 1975 potentially have an increased likelihood of fatigue issues. Fracture toughness requirements were also implemented in 1975. In 1994, the design specifications changed from load factor design (LFD) to load and resistance factor design (LRFD). This change was intended to increase reliability in bridge design. Consequently, bridges built prior to 1975 were not included in this family.

Fracture critical and scour critical bridges have established and separate inspection procedures from typical highway bridges. Fracture critical bridges require a hands-on inspection, and the inspection interval can be determined using the approach presented in the NCHRP 12-87 study, or can be determined using a calendar-based approach. Scour critical bridges require a plan of action to be developed to monitor and mitigate the damage. Because of the individualized nature of the required inspections, fracture critical and scour critical bridges were not included in this family.

Impact damage can increase the likelihood of failure by decreasing resistance to fracture for steel girders or compromising the concrete cover on concrete girders. Bridges previously impacted can also be assumed to have an increased probability of another impact. Vertical clearance is a key attribute relating to impact damage. Thus, bridges with impact damage were not included in this family.

Combining the occurrence factor and the consequence factor in the risk matrix determines the overall inspection interval. Bridges with the above criteria have a Low occurrence factor, and the consequence factor determines the inspection interval in accordance with the risk matrix, as shown in Table 6.1.

Table 6.1: Inspection Intervals for High Rated Family of Bridges

High Rated Family of Bridges		
Occurrence Factor	Consequence Factor	Inspection Interval
Low	Low	72 months
Low	Moderate	72 months
Low	High	48 months
Low	Severe	24 months

6.3.2. Low Rated

Another family of bridges consists of the low-rated bridges. Low rated bridges are those that have any component in condition rating CR 4 or below. These bridges exhibit an advanced rate of deterioration and are increasingly likely to reach a failed state within the inspection interval. Therefore, low rated bridges can be grouped together and assigned a 24 or 12 month inspection interval based upon the likelihood of failure. Bridges with the following characteristics are considered a part of this family:

- Deck <u>or</u> superstructure <u>or</u> substructure condition rating CR 4 or below
- Not fracture critical
- Not scour critical

6.3.3. Fatigue Susceptible Steel Bridges

Steel superstructure bridges with the following attributes are controlled by the damage mode of fatigue:

- Built before 1975

- Riveted or welded connections
- Fatigue Category D, E, or E' details
- ADTT > 500
- Finite fatigue life

Age is the first consideration for fatigue controlled assessments. Prior to 1975, fatigue design was based on principles not generally appropriate for welded structures. The modern fatigue provisions were incorporated into AASHTO Specifications in 1974. Therefore, bridges constructed before 1975 may be more susceptible to fatigue cracking than those constructed in or after 1975.

The Indiana inspection report notes whether bolts, rivets, or welds were used for connections. The fatigue category can be inferred from this data. For example, bridges with welded gusset plates have, at best, Category E details. Generally, poor fatigue details indicate bridges where fatigue cracks are more likely to develop.

The average daily truck traffic (ADTT) is a key attribute related to potential fatigue damage. For steel girders, research has shown that trucks produce the majority of fatigue damage in highway bridges. Therefore, bridges with high ADTT will accumulate fatigue damage at a faster rate than low ADTT bridges, and have a higher probability of fatigue damage. The Indiana RAP determined a cutoff of 500 trucks per day for use in determining the fatigue family.

Fatigue life is the last consideration to determine if fatigue considerations control the occurrence factor. If a bridge with the above attributes was designed to have an infinite fatigue life or was calculated to have an infinite fatigue life, the occurrence factor is Moderate. Bridges with a finite remaining fatigue life have a High occurrence factor.

Combining the occurrence factor and the consequence factor determines the overall interval. Based upon the risk matrix, a combination of Moderate occurrence factor and High consequence factor has a maximum overall inspection interval of 24 months. Bridges with a Moderate occurrence factor and a Moderate consequence factor would have a maximum overall interval of 48 months. In contrast, bridges with a High occurrence factor have a maximum inspection interval of 24 months for both Moderate and High consequence factors. Therefore, the majority of bridges in this family have a 24 month inspection interval.

6.3.4. SR 25 Hoosier Heartland Corridor

The Hoosier Heartland project created a new four-lane limited access highway linking Lafayette and Logansport to replace the existing two-lane rural highway. Bridges were constructed between 2009 and 2013. SR 25 overpass bridges can be grouped as a family, and SR 25 mainline bridges can be grouped according to feature under.

6.3.4.1. SR 25 Overpass Bridges

Bridges constructed during the Hoosier Heartland project over State Road 25 between Lafayette and Logansport can be considered a family. These bridges were constructed within a few years of each other using the same design characteristics, materials, construction processes, and span over a four-lane divided highway. A typical overpass bridge along SR 25 was built in 2009 or later and has three spans with a continuous prestressed concrete T-beam superstructure and concrete cast-in-place deck. The deck has epoxy coated reinforcing steel and a concrete wearing surface. It carries an average daily truck traffic of 100 vehicles per day, and has a maximum span length around 120 ft. Bridges with the following characteristics are in this family:

- Continuous prestressed concrete superstructure
- Concrete cast-in-place deck with epoxy coated reinforcing steel
- Spans SR 25
- Built in 2009 or after
- ADTT < 500
- No construction defects

The State Road 25 family also currently meets the High Rated family characteristics. Bridges are currently in condition rating CR 8 or CR 9, were built after 1975, are not fracture critical or scour critical, and have no impact damage. Therefore, the occurrence factor is Low. The INDOT determined consequence factor for redundant composite bridges over highways where falling debris may reach the roadway is High and applies to this scenario. Based upon these factors and the risk matrix, the risk-based inspection interval is 48 months.

6.3.4.2. SR 25 Mainline Bridges

Mainline bridges along the newly constructed SR 25 are varied in design, material type, and feature under. Superstructures range from steel girders to prestressed concrete I-beams to prestressed concrete T-beams. The mainline bridges also have a variety of features under including county roads, railroads, and waterways. It is impractical to group the SR 25 mainline bridges into a single family based upon the variation in feature under.

Bridges can however be grouped according to feature under. Mainline bridges over a roadway can be grouped together as a family. In the current condition, the bridges fall into the High Rated family as well, and have a Low occurrence factor. Because they are over a roadway, the consequence factor is High. Therefore, the overall inspection interval based upon the risk matrix is 48 months. Bridges in good condition spanning a waterway would have a Low occurrence factor and Moderate consequence factor. These can also be grouped as a family. Based upon the risk matrix, the inspection interval would be 72 months.

6.3.5. I-69 Southern Corridor

The I-69 project created a new four-lane limited access highway between Evansville and Indianapolis. Multiple overpass and mainline bridges were constructed beginning in 2009 and continuing through the present (2014). I-69 overpass bridges and I-69 mainline bridges can be considered as families.

6.3.5.1. I-69 Overpass Bridges

Bridges constructed over the I-69 corridor between Evansville and Indianapolis can be considered a family. These bridges were constructed within a few years of each other using the same design characteristics, materials, and construction processes, and span over a four-lane divided highway. A typical overpass bridge along I-69 was built in 2009 or later and has two spans with a continuous prestressed concrete T-beam superstructure and concrete cast-in-place deck. The deck has epoxy coated reinforcing steel and a concrete wearing surface. It carries an average daily truck traffic of less than 1000 vehicles per day, and has a maximum span length around 115 ft. Bridges with the following characteristics are considered a part of this family:

- Continuous prestressed concrete superstructure
- Concrete cast-in-place deck with epoxy coated reinforcing steel
- Stay-in-place forms present
- Spans I-69
- Built in 2009 or after
- ADTT < 1000
- No construction defects

The I-69 overpass bridges also currently meet the High Rated family characteristics. Bridges are in current condition rating CR 8 or CR 9, were built after 1975, are not fracture critical or scour critical, and have no impact damage. Therefore, the current occurrence factor is Low. The INDOT determined consequence factor for redundant composite bridges over highways where debris may reach the roadway is High and applies to this scenario. Stay-in-place forms prevent spalled concrete from the underside of the deck from falling onto the roadway; however, spalled concrete from the beams may reach the roadway. Based upon these factors and the risk matrix, the current risk-based inspection interval is 48 months.

6.3.5.2. I-69 Mainline Bridges

Mainline bridges along the newly constructed southern portion of I-69 can be considered a family. These bridges share many of the same attributes including date built, material types, traffic volume, and design features. A typical mainline bridge was built in 2009 or after and has a prestressed concrete T-beam superstructure. The deck is concrete cast-in-place with epoxy coated reinforcing steel. Maximum span lengths are between 100 and 150 feet, and the bridges carry around 2500 trucks per day. Bridges with the following characteristics are considered part of this family:

- Prestressed concrete T-beam superstructure
- Concrete cast-in-place deck with epoxy coated reinforcing steel
- Carries I-69
- Built in 2009 or after
- No construction defects

Mainline I-69 bridges also currently meet the characteristics for the High Rated family. The bridges are in current condition rating CR 8 or CR 9, and were built after 1975. In addition, the bridges are not fracture critical or scour critical and span over waterways, such that impact is not a concern. Therefore, the current occurrence factor is Low. The INDOT determined consequence factors for bridges carrying an interstate are either High or Severe. In the event of failure, the multi-girder redundant structures are expected to retain structural capacity. Short-term consequence of a lane closure may be High or Severe based upon the traffic volumes and the lane closure policy. With a Low occurrence factor and a High consequence factor, the risk matrix gives a maximum inspection interval of 48 months. For a Low occurrence factor and a Severe consequence factor, the inspection interval would be to be 24 months.

6.4. <u>Current Indiana Bridge Inventory Application</u>

An evaluation of the state-owned Indiana bridge inventory was conducted using data from the Bridge Inspection Application System (BIAS) database and the developed criteria for the families of bridges. Data used in this evaluation was from January 2014, with a total bridge count of 6,095 state-owned bridges in Indiana. Because the inventory is constantly evolving, with new bridges being added and inspection reports being updated, the presented values may not be exact. However, a general picture of the current bridge inventory can be clearly seen from the evaluation.

Fracture critical and scour critical bridges make up approximately 2% and 1%, respectively, of the state-owned Indiana bridge inventory. These bridges are effectively screened out from the risk methodology to account for the special considerations required to maintain safety and serviceability. Considerations include inspection type, mitigation plans, and the severe consequence of failure. A separate risk-based methodology for fracture critical and scour critical bridges could be implemented in the future.

The High Rated family of bridges includes 20% of the Indiana inventory. This value includes the SR 25 and I-69 families of bridges. Of these 20%, 7% have an inspection interval of 48 months. These are bridges with a Low likelihood factor and High consequence factor such as those over another roadway. The remaining 13% have an inspection interval of 72 months based upon a Low likelihood factor and a Low or Moderate consequence factor. A typical bridge in this category carries a low to moderate volume road over a waterway. Bridges in this family have favorable characteristics and are currently rated in good condition. In addition, these bridges fall into the useful life section of the bathtub model and, with proper maintenance, are expected to have multiple extended inspection intervals.

The Low Rated Family of bridges consists of approximately 4% of the Indiana inventory. Bridges in this family have advanced deterioration and a High likelihood of failure within the inspection interval. Therefore, bridges are assigned a 24 month interval to ensure safety and serviceability. Bridges in this family are in the wear-out portion of the bathtub curve. One goal of the risk methodology is to prevent bridges from reaching a low rated condition by identifying damage modes that require maintenance or repairs during the inspection. These specific areas can be addressed individually and the overall risk can be decreased.

The Fatigue Susceptible family of bridges consists of around 8% of the Indiana inventory. Bridges in this family have attributes that make the bridge susceptible to fatigue damage and have a High likelihood of failure. Therefore, the risk process recommends a 24 month inspection interval. During the inspection, inspectors need to check thoroughly for propagating fatigue cracks to maintain the safety of the bridge. In general, these are some of the older bridges in the inventory, and are connected using rivets or welds. Condition ratings range from poor to good, and as older bridges are rebuilt to modern standards, this family will shrink in size. Ultimately, the poor fatigue attributes, and not the condition rating, control the inspection interval for this family of bridges.

The SR 25 and I-69 families of bridges can be used as a case-study to evaluate the risk-based inspection procedures in real-time. The risk model could be implemented and evaluated for these bridges before being applied to the entire inventory, and any potential problems could be identified and solved. Inspections would still need to be performed in accordance with the biennial inspection law, but the risk procedure could be used simultaneously. For future communications with the public, these families can also be used as proof-of-concept to demonstrate the increase in safety, serviceability, and the optimization of inspection resources.

Overall, the risk methodology can have an immediate positive impact on bridge inspection intervals in Indiana. Approximately 20% of the inventory can have extended intervals based upon the performed family analysis and the risk methodology. These bridges are the easily identified as belonging in the High Rated family and can be considered the top part of the inventory. These bridges would have an inspection interval of 48 or 72 months. The middle part of the inventory consists of individual bridges that require assessment to determine inspection intervals. It is expected that many of these bridges will also have extended inspection intervals of 48 or 72 months, though some bridges may have a 24 month interval. The bottom 12% of the inventory consists of the Fatigue Susceptible and Low Rated Families, and bridges in this section have an inspection interval that would remain at 24 months. Based upon the case studies of sample bridges performed in Indiana, 21 of the 36 bridges evaluated had extended inspection intervals during their lifespan. If this trend carries over to the entire inventory, up to 60% of Indiana state-owned bridges could have extended intervals when the risk methodology is used. Indiana would immediately benefit from the implementation of a risk-based bridge inspection program.

6.5. Implementation

The implementation of the risk-based inspection procedures may be a challenge in the short-term, but has outweighing payoffs in terms of increased safety, increased reliability, and increased efficiency of inspections in the long term. Challenges that exist are political, organizational, and developmental in nature. An implementation strategy to provide a technical foundation for the methodology and develop community support is proposed to ease the transition period.

6.5.1.1. Implementation Challenges

Modifying the existing inspection system will present a political challenge. The current legislation requirements, including the CFRs, currently mandate a 24 month inspection interval with an option for 48 months. This prevents a risk-based methodology from being fully implemented, and will require modification. The technical audience is likely to recognize the benefits of a more rational system; however, the non-technical audience may be difficult to convince that decreasing the number of inspections for certain bridges will actually result in an overall increase in safety and serviceability. In addition, because the rate of deterioration is slow and failures rare, generating data to measure safety improvements will take time.

Risk-based inspections present an organizational challenge. Compared to a calendar based approach, risk-based inspection requires additional engineering to complete. Inspection personnel and organizational structures may need to be rearranged to better fit the new methodology. Reorganizing inspection reports to better reflect the information required for the new methodology may be necessary. Personnel with suitable experience and knowledge will also be required to effectively conduct the assessments. This will require training on the key elements of the methodology. Finally, a system that organizes the bridge inventory by required inspection date is needed to ensure bridges are inspected at the proper time.

Developing infrastructure and technology to support risk-based inspections is another potential challenge. Current inspection forms may require modification to include additional information needed for an effective risk assessment. New inspection programs may need to be developed to condense condition reports, track inspection intervals, and record comments. Technology can simplify the implementation of the risk methodology once developed.

6.5.1.2. Implementation Strategy

The strategy to implement risk-based inspection in Indiana consists of four steps: (1) perform additional case studies, (2) develop training modules, (3) develop communications strategies, and (4) develop software. On a national level, establishing an oversight committee can also aid in short and long-term implementation. Following these steps will help the transition to risk-based inspections from calendar based inspections and help gain widespread acceptance of the new methodology.

The first step in the implementation process would be to perform additional case studies. The 36 evaluated bridges in the study demonstrate the overall effectiveness of the risk-based procedure. Additional studies can fine-tune the procedure, test the application limits of the risk methodology, identify implementation challenges, and provide additional data on transitioning. Additional case studies would also provide baseline data and build further confidence in the procedure.

Developing training modules for RAP members and inspectors would be necessary for successful implementation of the risk methodology. Training modules and methods developed for the Indiana RAP meeting were proven to be effective, and provide a foundation for more formal training in the future. These modules include the theory and approach for RBI planning, deterioration and risk theory, and methodologies for expert elicitation. RAP members provide objective expertise on the local inventory away from political and management pressures (e.g., political pressure to extend intervals only to save money). It is imagined that inspector training will utilize the two-week training course put on by the National Highway Institute (NHI) as base training, and then include an additional segment for state-specific risk training. The NHI training modules and Indiana Bridge Inspection Manual will need to be updated to reflect the RBI procedures and provide an appropriate level of technical detail. The emphasis on damage modes is different from traditional defect detection practices, and inspectors may need to identify additional inspection items. For example, increased training to detect fatigue cracking may include proper lighting and distance requirements, and thoroughness of the inspection. Other techniques such as sounding could also be included. Modules that could be appended to the current two-week course that specifically relate to RBI are shown in Table 6.2.

Table 6.2: Proposed Training Modules for Inspectors

Module I: Background	
Topics	Material Covered
Deterioration Mechanisms for Bridges	Overview of typical deterioration patterns
Fundamentals of Risk Theory and Application to Inspection	Background overview of the underlying theories for RBI, risk matrices and likelihood
Risk Assessments for RBI	RAP process and basis for inspection procedures
Module II: Practices	
Understanding the IPN	Required thoroughness of inspection and prioritization of damage modes
Inspection Needs, Criteria, and Reporting	Focus and scope of inspections for RBI, access requirements, reassessment criteria, documentation and reporting requirements.
Enhanced Inspection Methods for RBI	Technologies and methods for detecting identified damage modes, enhanced methods for RBI, sounding and crack detection

Next, developing communications strategies between policy makers, INDOT officials, and the general public is a key element of the implementation plan. For the risk approach to be successful, the proposed risk methodology will need to be fully embraced. Policy makers can implement changes in the bridge inspection program, and can also block changes. The benefits of RBI will therefore need to be clearly communicated between INDOT and policy makers. Without proper communication about the risk methodology, a few potential challenges can be identified. There is potential for the reallocation of inspection resources to be seen as a cost saving measure instead of a measure to effectively ensure bridge safety. If viewed as a cost saving measure, budget cuts could lead to a reduction in inspection resources. Additionally, there may initially be some resistance to increasing inspection intervals because of historical precedent. It will clearly need to be explained that lengthening the intervals actually increases safety and serviceability by focusing inspection resources where they are most needed. Policy change will need to be communicated to the general public to retain trust in the safety of the nation's bridge systems. Non-technical publications could describe the approach, highlight the benefits, and demonstrate the increased safety and reliability aspects.

Another step in the implementation approach is software development and integration. Software that meets the needs of risk-based inspection processes can be tailored to integrate with the existing system and allow widespread implementation. The process for determining occurrence factor can be repetitive, and therefore lends itself well to software applications. Families of bridges can be easily identified based upon the input attributes. The development of software can also rapidly allow the methodology to be implemented, and will be essential for implementation efforts to successful.

On a national level, establishing an oversight committee to develop and maintain the risk methodology is an important element for short and long-term implementation. In the short term, the committee can aid in the transitioning process by sharing advice from states that have already implemented the methodology with states that have not yet transitioned. Transparency between states and agencies would be a key goal. The committee can also have a long-term commitment to maintain and further develop the guidelines, as is common with many design codes. Members of the committee should be diverse and include representatives from different regions and different types of bridge inventories. Inclusion of the FHWA on this committee would also be desirable. Other goals of the committee could be identifying research goals and making changes to the risk methodology as needed.

6.6. Summary

Families of bridges have similar damage modes and are expected to deteriorate in the same fashion at nearly the same rate. Proposed families for the Indiana state-owned bridge inventory include the High Rated family, the Low Rated Family, the Fatigue Susceptible family, the SR 25 families, and I-69 families. An evaluation of 36 bridges in the Indiana inventory was conducted, and 21 of those 36 bridges (60%) had extended inspection intervals at some point during the lifetime of the bridge. Implementation of the risk-based inspection practices will need to overcome political, organizational, and developmental challenges. However, with the proposed implementation plan, the payoffs of increased safety, increased reliability, and increased optimization of inspection resources are well within reach.

CHAPTER 7. RESULTS, CONCLUSIONS, & FUTURE RESEARCH

7.1. Results

The risk-based bridge inspection procedure proposed in NCHRP 12-82 (Washer & Connor, 2014) was customized for the state of Indiana. This included the development of guidelines through the use of expert elicitation for the occurrence factor, consequence factor, and attributes for Indiana bridges. The results of the expert elicitation from the Indiana RAP meeting can be found in Appendix A. Consequence factor guidelines and tables can be found in Appendix B. The benefits and challenges of implementing a risk-based inspection procedure in Indiana were also investigated.

The Indiana specific risk-based methodology was evaluated using historical inspection records in a procedure called "back-casting." Back-casting involved monitoring deterioration progression through historical data, and then comparing the results with the risk approach. Thirty-six randomly selected bridges from diverse geographical locations and superstructure types in Indiana were evaluated using the back-casting procedure. Appendix C contains the results.

Families of bridges were developed for the Indiana bridge inventory to recognize similarity of design, condition, and loading attributes in the risk process. These families included High Rated, Low Rated, Fatigue Susceptible, SR 25, and I-69 bridges. Each family was selected based upon similar damage modes, characteristics, and expected deterioration patterns. Analysis of the Indiana inventory was conducted to determine inspection intervals for the families and to determine the number of bridges in each family.

Training on how to use the RBI system and a proposed implementation plan was also provided to Indiana. On-site training occurred during the Indiana Risk Assessment Panel (RAP) meeting held October 23-24, 2013 in Indianapolis, Indiana. Powerpoints explaining the concepts and procedures, as well as workshop booklets and packets were created and presented to INDOT officials and consultants. The training guided the development of Indiana specific damage modes, bridge attributes, and consequence factors. An implementation plan was also suggested.

7.2. Conclusions

Key conclusions that can be drawn as a result of the risk-based bridge inspection practices study are:

- Bridge inspection intervals of 48 and 72 months are suitable for typical highway bridges in Indiana. The longer intervals did not adversely affect safety and serviceability based upon the analysis of historical bridge inspection records.

- Expert elicitation in the form of a Risk Assessment Panel (RAP) comprised of state and industry experts familiar with the bridge inventory is an effective method for determining damage modes, attributes, and consequence factors.

- Criteria for risk-based inspections were developed in Indiana including the determination of damage modes, attributes, and consequence factors for steel, reinforced concrete, and prestressed superstructure bridges as well as reinforced concrete decks and substructures.

- Families of bridges for the Indiana inventory were created and include High Rated, Low Rated, Fatigue Susceptible, SR 25 bridges, and I-69 bridges. Families make the RBI process more efficient by grouping bridges of similar design, loading, and condition characteristics that are expected to deteriorate in the same manner at nearly the same rate.

- Indiana can immediately benefit from the implementation of risk-based inspection practices. Based upon families of bridges, 20% of the Indiana inventory can have extended intervals of either 48 or 72 months.

- During the back-casting evaluation, there were no cases where a bridge deteriorated to a serious condition during the hypothetical proposed extended inspection intervals.

- Of the 36 bridges analyzed during the back-casting process, 21 had extended intervals at some point during their lifespan.

7.3. Future Research

There are three main recommendations for future research. First, it is recommended that back-casting case studies are conducted in additional states across the country to prove the process across more bridge populations, different families, and various owners. The risk methodology can be further verified and any issues that arise can be addressed. This will also aid in credibility when implementing risk procedures on a large scale. The second recommendation for future research is the development of computer software for the RBI process. Specialized software will enable risk-based procedures to integrate with the current databases and assist with widespread implementation. A third suggestion for future research would be to develop a risk procedure for atypical bridges including non-redundant members, complex bridges, bridges with advanced deterioration, and bridges with MSE walls.

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APPENDICES

Appendix A: Indiana Rap Meeting Results

Summary of the RAP Meeting

On October 23rd and 24th, 2014 a Risk Assessment Panel (RAP) workshop for Indiana bridges was held at the INDOT office located in Indianapolis, Indiana. Discussion centered on identifying key attributes in INDOT's bridge inventory to better implement Risk-Based Inspection (RBI) methodologies. The RBI methodology is a risk-based bridge inspection practice with the potential for setting inspection intervals from 24 to 72 months based on a rational risk-based methodology. The methodologies were originally developed through NCHRP Project 12-82, *Developing Risk-Based Bridge Inspection Practices,* recently completed by Dr. Glenn Washer of University of Missouri and Dr. Robert Connor of Purdue University.

Discussion on the first day of the workshop centered on likelihood analysis. Participants listed possible damage modes for decks, steel superstructures, and prestressed superstructures. Examples of damage modes for decks included corrosion and cracking. The panel also determined attributes of a bridge that would lead to the damage modes, and ranked them according to severity. The results are shown in the Likelihood Analysis section of this summary. Discussion and notes follows the tables for specific topics addressed at the workshop.

Discussion on the second day of the workshop centered on consequence analysis. Each participant of the panel ranked the consequence for the given damage modes such as deck cracking and prestressed strand corrosion. Based upon the responses, a consensus was reached upon the consequence of each damage mode. The results can be seen in the tables of the Consequence Analysis section of this summary. A discussion also occurred about the possibility of integrating Indiana's "Interstate Congestion Policy" with the consequence factors.

Discussion on Likelihood

The following tables contain the summary of results for the discussion on likelihood from the Indiana RAP meeting. Corrosion in concrete decks, section loss and fatigue cracking in steel superstructures, strand corrosion, steel reinforcing bar corrosion, shear cracking and bearing seat issues in prestressed superstructures, and fire incidents and flooding incidents were discussed and have summary tables contained in this appendix.

The first column in the table describes attributes that are similar to ones used in the NCHRP 12-82 study. The attribute is described in the second column. The third through seventh columns define the different risk levels for that attribute. For example, a condition rating for a bridge element of CR 5 is high because it is perceived as the least reliable. A condition rating of 7 or above is considered more reliable and is therefore located in the low column. The point value for determining the occurrence factor is also dependent upon the high to remote breakdown, with high attributes receiving the most points. The degree of severity is determined by the RAP and is represented where H = high, M = moderate, and L = low. The max score correlates to the degree of severity. The max score for a high degree of severity is 20 points, while the max score for a moderate severity is 15 points. Low degree of severity have a max score of 10 points.

For example, in the deck/corrosion table efflorescence & leaching is an attribute listed. It is similar to the condition attribute C.13 found in NCHRP 12-82. The degree of severity for this attribute was determined to be moderate. Therefore, the max score is 15 points. To develop the point system for the occurrence factor, efflorescence with rust staining was considered to have the highest likelihood of failure, or the least reliable condition. Bridges exhibiting this condition are given 15 points. On the other end of the scale, bridges with no efflorescence are considered to have a remote possibility of failure, and are assigned 0 points. To fill in the middle, moderate efflorescence is assigned 10 points, while bridges exhibiting minor efflorescence without rust staining are assigned 5 points. This process was completed for each attribute and the point framework for determining inspection interval developed.

Deck / Corrosion

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
C.1	Current Deck Condition	CR 5	CR 6	CR 7+			H	20
C.11	Presence of Repairs	Yes			No		L	10
C.13	Efflorescence/ Leaching	Efflorescence with rust staining	Moderate Efflor.	Minor Efflor. without rust staining	No Efflor.		M	15
C.5	Maintenance Cycle	No maintenance			Washing / Sealing		H	20
D.11	Concrete Cover	<1.5"	1.5" - 2.5"	2.5"+			M	15
D.12	Reinforcement Type	Not Epoxy Coated			Epoxy Coated		M	15
D.4	Deck Drainage	Ponding/ Ineffective Drainage			Effective Drainage		M	15
D.7	Presence of Overlay/Type	Bituminous without Membrane			No Overlay or LMC overlay		M	15
L.1	ADTT (Functional Class)	>2500 -- Interstate			<100 -- Rural		M	15
L.3	Exposure Environment	Northern Districts	Central Districts	Southern Districts			H	20
-	Composite with Superstructure					X		
-	Construction Error					X		

Discussion & Notes:

1. Screening: Composite with Superstructure – screen out bridges with non-composite decks.
2. Screening: Construction Error – screen out bridges with a known construction error.

3. INDOT does not typically place asphalt overlay unless deck is scheduled to be replaced. Asphalt overlay is considered to be unfavorable.
4. Current testing is being performed on the Toll Road with torch applied membranes and asphalt overlays.
5. Maintenance – cleaning the shoulders and joints is believed to be more effective than bridge washing.

Steel Superstructure / Section Loss

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
-	Type of Deck					X	S	
C.17	Coating Type/ Condition	No coating/ Ineffective			Effective Coating		M	15
C.21	Existing Section Loss	Significant amount of corrosion	Moderate amount of corrosion	Minor amount of corrosion	No active corrosion	X	H	20
C.4	Adequate Drainage	Drains onto superstructure			Adequate Drainage		M	15
C.5	Maintenance Cycle	No maintenance			Regular Maintenance		M	15
C.7	Condition of Joints	Open Joints/Failed Joints	Leaky Joints	New Joints	Jointless Bridge		H	20
D.6	Year of Construction		2000 or before	2000+			M	15
L.1	ADTT (Functional Class)	>2500			<100		L	10
L.3	Exposure Environment	Northern Districts	Central Districts	Southern Districts			H	20

Discussion & Notes:

1. Screening – Type of Deck – screen out bridges with timber or open decks.
2. Screening – Existing Section Loss – screen out if severe section loss is present.

Steel Superstructure / Fatigue Cracking

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
C.18	Existing Fatigue Cracks	Yes			No		H	20
C.18	Presence of Repaired Cracks	Yes			No		H	20
C.18	Existing Distortion Induced Cracks	Yes			No		H	20
D.16	Fatigue Detail	E/ E'	D		C / B / A		H	20
D.6	Year of Construction	<1975	1976-1984	1985-1993	1994+		M	15
L.1	ADTT (Functional Class)	>2500			<100		H	20

Discussion & Notes:

1. Bridges with web gaps are important to track. 90% of these bridges end up forming a fatigue crack.
2. Remaining life calculation may be overly detailed for this type of approach; therefore is was not considered an attribute.
3. Fatigue Detail – Categories C/B/A are remote because experience shows Category C has not presented problems in the past.

Prestressed Superstructure / Strand Corrosion

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
C.1	Current Superstructure Condition	CR 4 or less	CR 5/6	CR 7+			H	20
C.8	Existing Corrosion Damage	Significant amount of corrosion	Moderate amount of corrosion	Minor amount of corrosion	No active corrosion	X	H	20
D.11	Concrete Cover	<1.5"	1.5" - 2.5"	2.5"+			H	20
D.12	Reinforcement Type	Not Epoxy Coated			Epoxy Coated		M	15
D.18*	Bad End Detail	Strand Exposed to Environment			Not Exposed to Environment		L	10
L.3	Exposure Environment	Northern Districts	Central Districts	Southern Districts			H	20

Discussion & Notes:

1. Screening – Existing Corrosion Damage – screen out bridges with severe corrosion damage
2. Potential Screening Criteria – Bridges with delayed ettringite formation (DEF). Poor materials with a lot of cracking may need to be replaced immediately.
3. Potential Screening Criteria – Bridges with adjacent box beams superstructure.

Prestressed Superstructure / Rebar Corrosion within the Span

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
L.3	Exposure Environment	Northern Districts	Central Districts	Southern Districts			H	20
C.6 and C.21	Previously Impacted & Active Corrosion	Collision Damage: Severity 4			Collision Damage: Severity 0	X	H	20
D.11	Concrete Cover	<1.5"	1.5" - 2.5"	2.5"+			H	20
D.12	Reinforcement Type	Not Epoxy Coated			Epoxy Coated		M	15

Discussion & Notes:

1. Potential screening criteria: bridges that have been impacted repeatedly.
2. Epoxy coated rebar has the potential to be damaged when placed. This would limit effectiveness.
3. INDOT has not previously coated prestressing strands. Having both prestressing strands and reinforcement epoxy coated is unfavorable.

Prestressed Superstructure / Shear Cracking

Similar items in NCHRP 12-82	Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score
D.2	Load Posting	Posted			Not Posted		H	20
D.6	Year of Construction		<2000	>2000			L	10
L.4	Likelihood of Overload	High Likelihood		Low Likelihood			L	10

Discussion & Notes:

1. The criteria proposed by Oregon DOT for shear cracking was found to also apply to Indiana.
2. Likelihood of overload is typically determined by identifying roads where permit loads travel.

Prestressed Superstructure / Bearing Seat Issues

1. INDOT has never adjusted inspection cycles based upon bearing seat issues, therefore this damage mode has been removed from consideration.
2. Bearings are expected to last the life of the bridge.
3. Elastomeric pads are replaced on occasion. Some maintenance on bearings is performed as well.

Fire Incident

1. Inspect immediately after event and 6 months after event to check for damage.
2. Return to routine inspection cycle if no damage/cracking discovered.

Flooding Incident

1. Inspection cycle continues as normal, unless noticeable damage occurs.
2. If the road is closed, inspectors will do a visual check to ensure safety before reopening.

Discussion on Consequence

The following tables present the results of the Indiana RAP consequence discussion. The specific consequence scenarios addressed were overlay debonding, deck spalling, steel girder cracking, prestressed strand corrosion, fascia girder damage, and pier corrosion. Each participant ranked what he or she thought the proper consequence should be for each scenario. Participants could place 100% on one category or could divide their 100% into separate categories using 10% increments. After compiling the results, the consensus consequence factor was determined by averaging the results. In many cases, the consequence factor was clear. In other cases, the RAP was divided between two separate consequence factors. For those cases, it was determined that the consequence factor relied on the operating conditions of the bridge. For example, the deck spalling consequence scenario was tied between a Moderate and High consequence factor. The RAP determined that the traffic volume was a key attribute that would determine whether a Moderate or High consequence factor was appropriate, with higher traffic volumes having a High consequence factor.

A general discussion on consequence also occurred. Two main points stemmed from this discussion:

1. The “Interstate Congestion Policy” would be a useful tool to help determine the short-term consequence. The effect of a lane/shoulder closure is included in this policy and could aid in separating the High consequence bridges from the Moderate consequence bridges based upon effect to traffic. Then, all interstate bridges don’t need to be ranked together, since some interstates in Indiana carry more traffic than others.

2. The “Interstate Congestion Policy” could also be a useful tool for inspections. If a bridge requires nighttime or weekend closures based upon the policy, a longer inspection cycle could be beneficial.

Overlay Debonding					
		Low	**Moderate**	**High**	**Severe**
1	Participant 1	100	-	-	-
2	Participant 2	90	10	-	-
3	Participant 3	-	10	50	40
4	Participant 4	60	30	10	-
5	Participant 5	10	70	20	-
6	Participant 6	-	100	-	-
7	Participant 7	40	50	10	-
8	Participant 8	40	60	-	-
9	Participant 9	50	50	-	-
10	Participant 10	80	20	-	-
11	Participant 11	50	40	10	-
		0.47	**0.40**	**0.09**	**0.04**
The Consequence of this Damage Mode is Low.					

Deck Spalling Consequence					
		Low	**Moderate**	**High**	**Severe**
1	Participant 1	90	10	-	-
2	Participant 2	30	40	20	10
3	Participant 3	-	50	50	-
4	Participant 4	-	10	40	50
5	Participant 5	-	40	60	-
6	Participant 6	-	90	10	-
7	Participant 7	10	50	40	-
8	Participant 8	-	40	60	-
9	Participant 9	-	20	80	-
		0.14	**0.39**	**0.40**	**0.07**
The Consequence of this Damage Mode is Moderate/High.					

Steel Girder Cracking Consequence					
		Low	Moderate	High	Severe
1	Participant 1	10	30	50	10
2	Participant 2	-	-	80	20
3	Participant 3	10	60	30	-
4	Participant 4	-	30	60	10
5	Participant 5	10	70	10	10
6	Participant 6	-	30	70	-
7	Participant 7	-	30	70	-
8	Participant 8	20	50	30	-
9	Participant 9	20	50	30	-
		0.08	**0.39**	**0.48**	**0.06**
The Consequence of this Damage Mode is High.					

Prestressed Strand Corrosion					
		Low	Moderate	High	Severe
1	Participant 1	-	-	50	50
2	Participant 2	-	-	-	100
3	Participant 3	-	20	50	30
4	Participant 4	-	0	70	30
5	Participant 5	-	40	60	-
6	Participant 6	-	10	80	10
7	Participant 7	-	30	60	10
8	Participant 8	-	40	50	10
9	Participant 9	-	40	60	-
10	Participant 10	-	30	70	-
11	Participant 11	-	30	50	20
		0.00	**0.22**	**0.55**	**0.24**
The Consequence of this Damage Mode is High.					

Fascia Girder Damage					
		Low	**Moderate**	**High**	**Severe**
1	Participant 1	-	-	50	50
2	Participant 2	-	-	50	50
3	Participant 3	-	-	60	40
4	Participant 4	-	-	-	100
5	Participant 5	-	10	60	30
6	Participant 6	-	10	90	-
7	Participant 7	-	10	70	20
8	Participant 8	-	40	50	10
9	Participant 9	-	50	50	-
10	Participant 10	-	-	100	-
11	Participant 11	-	-	70	30
		0.00	**0.11**	**0.59**	**0.30**
The Consequence of this Damage Mode is High.					

Pier Corrosion					
		Low	**Moderate**	**High**	**Severe**
1	Participant 1	-	50	50	-
2	Participant 2	90	10	-	-
3	Participant 3	50	30	20	-
4	Participant 4	-	-	80	20
5	Participant 5	-	70	30	-
6	Participant 6	-	60	40	-
7	Participant 7	10	30	50	10
8	Participant 8	10	40	40	10
9	Participant 9	-	100	-	-
10	Participant 10	-	50	50	-
11	Participant 11	60	30	10	-
		0.20	**0.43**	**0.34**	**0.04**
The Consequence of this Damage Mode is Moderate.					

Appendix B: Consequence Tables

Consequence Factor Tables

Further guidance for the consequence factor is presented in the following tables. For the deck and substructure consequence table, the assumed worst-case damage mode was spalling. The superstructure tables all assume loss of a primary load bearing member as their scenario. Descriptions for the immediate and short-term consequence for each consequence category—Low, Moderate, High, and Severe—are presented as well as sample situations where the category may apply and additional factors to consider.

Immediate Consequence

The immediate consequence refers to the structural integrity and safety of traveling public when the failure occurs. Considerations include whether a bridge will remain standing and whether the traveling public will remain safe. For example, failure of a load bearing member in a multi-girder redundant bridge is not expected to cause loss of structural integrity, excess deflections, or collapse. As a result, the traveling public is immediately unaffected when the failure occurs. A contrasting scenario would be for a fracture critical bridge, where the loss of a main member could cause excess deflection or collapse thereby causing the bridge to be immediately unsafe for the traveling public. The safety of the structure and the public should be considered for determining the immediate consequence. The primary considerations for determining immediate consequence are structural integrity and public safety.

Short-Term Consequence

The short-term consequence refers to serviceability concerns and short-term impacts to the traveling public after a failure occurs. Load posting, repairs, and speed reductions can be considered serviceability concerns. Lane, sidewalk, or shoulder closures as a result of the damage mode impact the traveling public and can cause delays. For example, a multi-girder redundant bridge that experiences the loss of a load bearing member is expected to remain standing; however, once the failure is discovered, a typical response is to close a lane or shoulder until the bridge is repaired. Therefore, the traveling public will be affected. The effect of a lane closure for a bridge carrying an interstate will have a higher short-term consequence than a rural bridge carrying a low traffic volume. Additionally, lane closures or speed reductions for bridges located in downtown regions or bridges that are critical links to towns can cause a large impact on traveling public. The primary considerations for determining short-term consequence are serviceability concerns and impacts to the traveling public.

Sample Situations

The sample situations column illustrates specific cases where the consequence factor may be appropriate. These situations are general guidelines only, and are not firm rules. Engineering experience and judgment should be applied to the specific conditions at each bridge to determine the appropriate consequence factor. Additional situation not described in this column will also apply to the specific consequence factor.

Factors to Consider

Multiple criteria exist for determining the immediate and short-term consequence factor. For some bridges, the consequence factor is clear, but for other bridges in-depth consideration is required. Some factors to consider when determining the consequence factor are ADT/ADTT, feature under, feature carried, presence of stay-in-place forms, redundancy, composite action, and load carrying capacity.

General Consequence Factor Table

The table below shows the overarching themes of the consequence factor determination. Safety and serviceability of the bridge are the primary concerns.

Level	Category	Consequence Description
1	Low	Minor effect on serviceability, no effect on safety
2	Moderate	Moderate effect on serviceability, minor effect on safety
3	High	Major effect on serviceability, moderate effect on safety
4	Severe	Major effect on safety and serviceability

Deck Consequence Table

Assumed damage mode is spalling

Consequence for Deck	Description	Sample Situations	Factors to Consider
Low	**Immediate:** Damage to the top of the deck does not present a safety concern for the traveling public. Falling debris from the bottom of deck does not affect the safety of the public. **Short-term:** Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.	• Bridge carrying low volume and/or low speed roadway • Bridge with concrete deck over a non-navigable waterway or unused right-of-way land	• ADT/ADTT • Feature under • Feature carried • Stay-in-place forms
Moderate	**Immediate:** Damage to the top of the deck presents a minimal safety concern to the traveling public. Falling debris from the bottom of deck presents a minimal safety concern. **Short-term:** Moderate serviceability concerns. Speed reduction may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Moderately traveled roadway where damage would cause minimal delays • Bridge with stay-in-place forms over roadway where spalls would not reach roadway or waterway	
High	**Immediate:** Damage to the top of the deck presents a moderate safety concern to the traveling public. Falling debris from the bottom of deck presents a moderate safety concern. **Short-term:** Major serviceability concerns. Repairs or speed reduction may be required. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• High volume roadway where damage would cause reduction in posted speed or potential for loss of vehicular control • Bridge without stay-in-place forms over heavily traveled waterway or high volume roadway	
Severe	**Immediate:** Damage to the top of the deck presents a major safety concern to the traveling public. Falling debris presents a major safety concern. Possible loss of life. **Short-term:** Potential for significant traffic delays on or under the bridge.	• Bridge over feature where spalling concrete would result in lane closure, loss of life, or major traffic delays	

Steel Superstructure Consequence Table

Assumed damage mode is loss of one primary load carrying member

Consequence for Steel Superstructure	Description	Sample Situations	Factors to Consider
Low	**Immediate:** Little to no impact on structural capacity is expected based upon structural analysis or documented experience. Public safety is unaffected. **Short-term:** Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.	• Bridge over non-navigable waterway or unused right-of-way land • Rural bridge with low ADT/ADTT	• ADT/ADTT • Feature under • Feature carried • Redundancy • Composite action • Load carrying capacity
Moderate	**Immediate:** Structural capacity is expected to remain adequate based upon structural analysis or documented experience. **Short-term:** Moderate serviceability concerns. Speed reduction or load posting may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Bridge over multi-use path, railroad or lightly traveled waterway • Bridge on or over moderate volume urban roadway or high volume rural roadway that would cause moderate delays for drivers	
High	**Immediate:** Structural capacity is expected to remain adequate. **Short-term:** Major serviceability concerns. Load posting, repairs, or speed reduction may be needed. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Bridge with alternate load path(s) that has an expectation of adequate remaining structural capacity • Lane or shoulder closure on or under roadway that would cause major delays for drivers	
Severe	**Immediate:** Structural collapse. Possible loss of life. **Short-term:** Potential for significant traffic delays on or under bridge.	• Bridge with high ADT/ADTT that requires closure	

Reinforced Concrete Superstructure Consequence Table

Assumed damage modes are loss of one primary load carrying member and/or spalling

Consequence for Concrete Superstructure	Description	Sample Situations	Factors to Consider
Low	**Immediate:** Little to no impact on structural capacity is expected based upon structural analysis or documented experience. Falling debris does not affect the safety of the public. **Short-term:** Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.	• Bridge over non-navigable waterway or unused right-of-way land • Rural bridge with low ADT/ADTT	• ADT/ADTT • Feature under • Feature carried • Redundancy • Composite action • Load carrying capacity
Moderate	**Immediate:** Structural capacity is expected to remain adequate based upon structural analysis or documented experience. Falling debris presents a minimal safety concern to the public. **Short-term:** Moderate serviceability concerns. Speed reduction or load posting may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Bridge over multi-use path, railroad or lightly traveled waterway • Bridge on or over moderate volume urban roadway or high volume rural roadway that would cause moderate delays for drivers	
High	**Immediate:** Structural capacity is expected to remain adequate. Falling debris presents a moderate safety concern to the public. **Short-term:** Major serviceability concerns. Load posting, repairs, or speed reduction may be needed. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Bridge with alternate load path(s) that has an expectation of adequate remaining structural capacity • Lane or shoulder closure on or under roadway that would cause major delays for drivers	
Severe	**Immediate:** Structural collapse. Falling debris presents a major safety concern to the public. Possible loss of life. **Short-term:** Potential for significant traffic delays on or under bridge.	• Bridge over feature where spalling concrete would result in lane closure, loss of life, or significant traffic delays	

Prestressed Concrete Superstructure Consequence Table

Assumed damage modes are loss of one primary load carrying member and/or spalling

Consequence for PS Superstructure	Description	Sample Situations	Factors to Consider
Low	**Immediate:** Little to no impact on structural capacity is expected based upon structural analysis or documented experience. Falling debris does not affect the safety of the public. **Short-term:** Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.	• Bridge over non-navigable waterway or unused right-of-way land • Rural bridge with low ADT/ADTT	• ADT/ADTT • Feature under • Feature carried • Redundancy • Composite action • Load carrying capacity
Moderate	**Immediate:** Structural capacity is expected to remain adequate based upon structural analysis or documented experience. Falling debris presents a minimal safety concern to the public. **Short-term:** Moderate serviceability concerns. Speed reduction or load posting may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Bridge over multi-use path, railroad or lightly traveled waterway • Bridge on or over moderate volume urban roadway or high volume rural roadway that would cause moderate delays for drivers	
High	**Immediate:** Structural capacity is expected to remain adequate. Falling debris presents a moderate safety concern to the public. **Short-term:** Major serviceability concerns. Load posting, repairs, or speed reduction may be needed. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Bridge with alternate load path(s) that has an expectation of adequate remaining structural capacity • Lane or shoulder closure on or under roadway that would cause major delays for drivers	
Severe	**Immediate:** Structural collapse. Falling debris presents a major safety concern to the public. Possible loss of life. **Short-term:** Potential for significant traffic delays on or under bridge.	• Bridge over feature where spalling concrete may result in lane closure, loss of life, or significant traffic delays	

Substructure Consequence Table
Assumed damage mode is spalling

Consequence for Substructure	Description	Sample Situations	Factors to Consider
Low	**Immediate:** Falling debris does not affect the safety of the public. Structural capacity of the bridge remains adequate. **Short-term:** Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.	• Bridge over non-navigable waterway or unused right-of-way land	• ADT/ADTT • Feature under • Load carrying capacity
Moderate	**Immediate:** Falling debris from substructure presents a minimal safety concern to the public. Structural capacity is expected to remain adequate based upon structural analysis or documented experience. **Short-term:** Moderate serviceability concerns. Speed reduction or load posting may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Bridge over multi-use path, railroad or lightly traveled waterway	
High	**Immediate:** Falling debris from substructure presents a moderate safety concern to the public. Structural capacity is expected to remain adequate. **Short-term:** Major serviceability concerns. Load posting, repairs or speed reduction may be needed. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Lane or shoulder closure on roadway that would cause major delays for drivers	
Severe	**Immediate:** Structural collapse, bearing area failure, or loss of load carrying capacity. Falling debris presents a major safety concern to the public. Possible loss of life. **Short-term:** Potential for significant traffic delays on or under bridge.	• Bridge adjacent to high volume roadway where spalling concrete may result in lane closure, loss of life, or major traffic delays • Bearing area failure resulting in deck misalignment	

Appendix C: Indiana Back-Casting Case Studies

To evaluate whether the risk-based procedure could establish a safe and effective inspection interval, a process called back-casting was performed. Back-casting involved monitoring deterioration progression through historical data, and then comparing the results with the risk approach. Thirty-six bridges in Indiana were assessed, and the results are presented in this appendix.

On the left side of the sheet, general information about each bridge is listed. Information includes attributes and characteristics, as well as the controlling damage mode. The bridge number is located at the top center of each sheet.

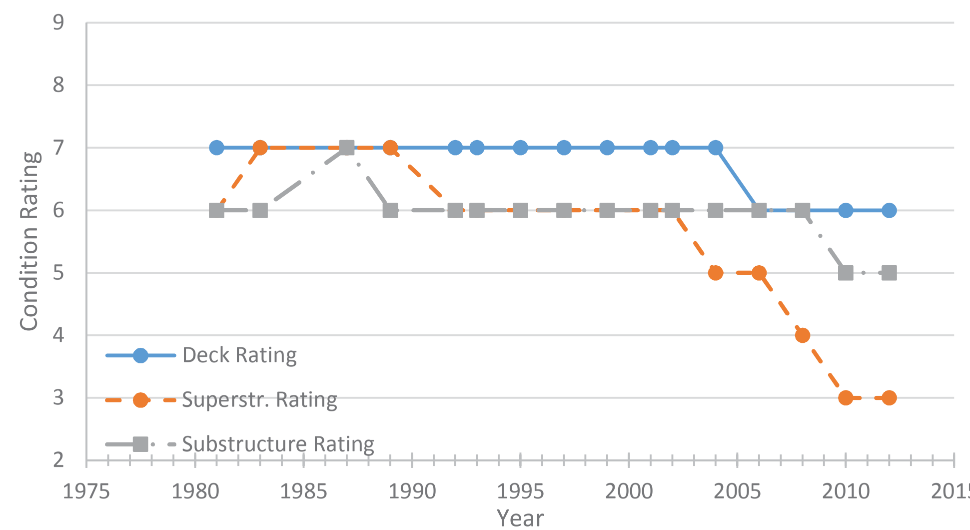
The graph in the right corner of each sheet tracks the condition rating of each bridge component—deck, superstructure, and substructure—over the life of the bridge. Typically, the condition rating decreases as the bridges ages and deteriorates. In some cases, the condition rating increases which corresponds to a repair to the bridge or a correction in condition rating by the inspector.

The determined inspection intervals using the risk-based inspection process are located underneath the general information and condition rating graph. An interval was determined for every year historical inspection data was available. For certain bridges, missing historical inspection records result in a gaps or an inconsistent interval on the timeline; however, these omissions were not considered critical. Evaluating intervals for every year data was available also demonstrates that the risk approach can be applied with any starting point.

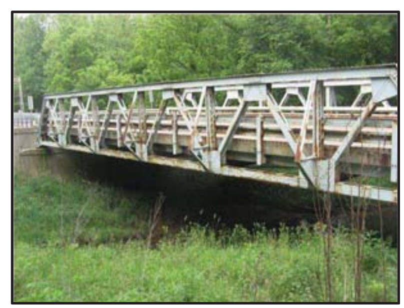
A representative sample of thirty-six Indiana bridges was considered, and there were no cases where a serious progression of damage would have been missed as a result of an extended interval. Fourteen of the thirty-six bridges had an inspection interval of 72 months at some point during the back-casting process and twenty-one bridges had a 48 month interval at some point during the process. Bridges in poor condition were assigned inspection intervals of 24 months. In addition, no unexpected or sudden changes to the NBI condition rating were noted during the risk-based inspection interval.

Bridge ID: (3)203-72-3236 A

Year Built	1939
No. of Spans	1
Bridge Length (ft.)	88.0
Max. Span Length (ft.)	84.0
Superstructure Type	SPT
ADT	2100
ADTT	10%
No. of Lanes	2
Location	Scott County
Environment	Southern District
Deck CR (2013)	6
Superstructure CR (2013)	3
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew	28
Controlling Damage Mode	Fatigue and Corrosion

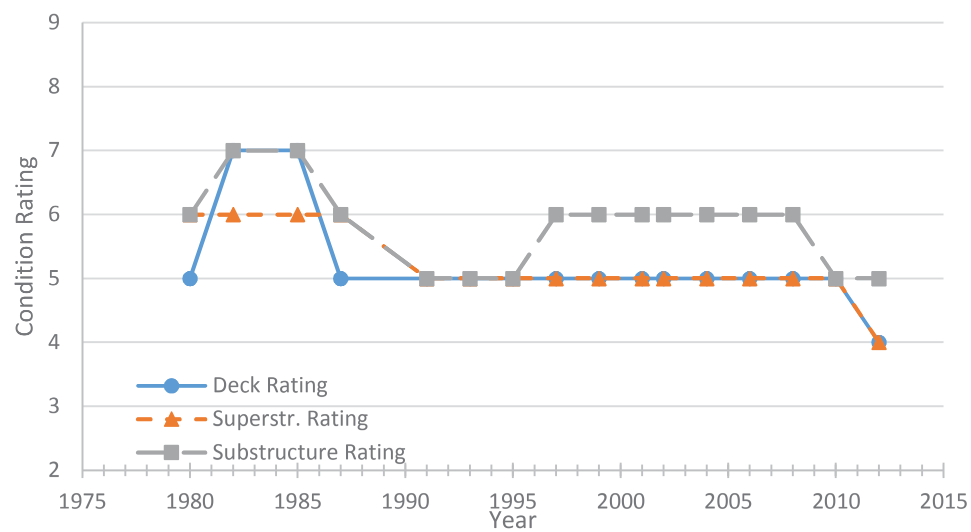


Year	1981	1983	1987	1989	1992	1993	1995	1997	1999	2001
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2002	2004	2006	2008	2010	2012				
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months				

Bridge ID: (237)37-14-01457

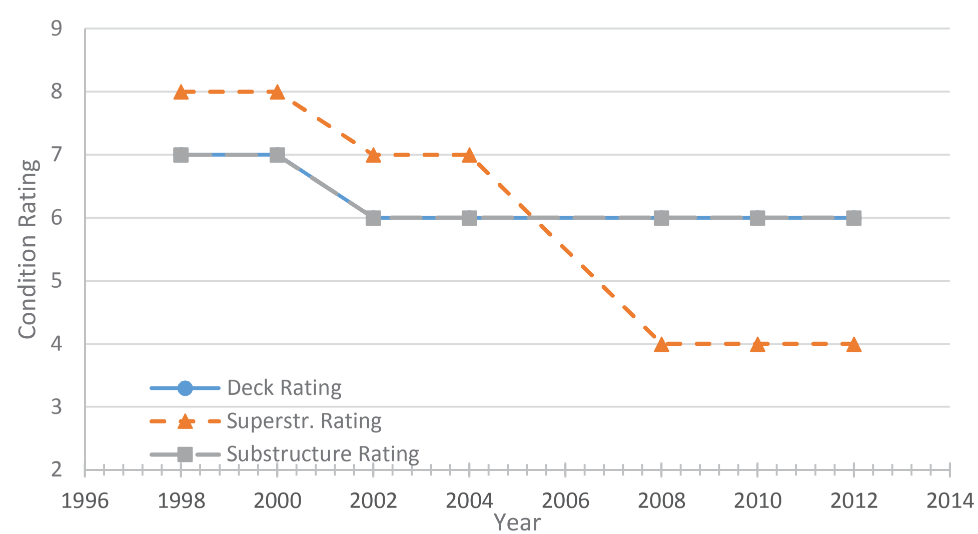
Year Built	1935
No. of Spans	3
Bridge Length (ft.)	77.0
Max. Span Length (ft.)	24.0
Superstructure Type	RCS
ADT	3600
ADTT	5%
No. of Lanes	2
Location	Crawford County
Environment	Southern District
Deck CR (2013)	4
Superstructure CR (2013)	4
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew	15
Controlling Damage Mode	Corrosion



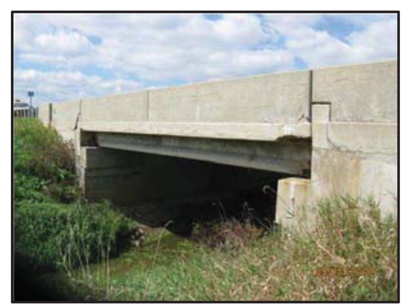
Year	1980	1982	1985	1987	1991	1993	1995	1997	1999	2001
Inspection Interval	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months
Year	2002	2004	2006	2008	2010	2012				
Inspection Interval	48 months	48 months	48 months	48 months	48 months	24 months				

Bridge ID: 06-20-01215 B

Year Built	1931
No. of Spans	1
Bridge Length (ft.)	38
Max. Span Length (ft.)	32
Superstructure Type	PCIB
ADT	8900
ADTT	10%
No. of Lanes	2
Location	Elkhart County
Environment	Northern District
Deck CR (2013)	6
Superstructure CR (2013)	4
Substructure CR (2013)	6
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew	15
Controlling Damage Mode	Corrosion

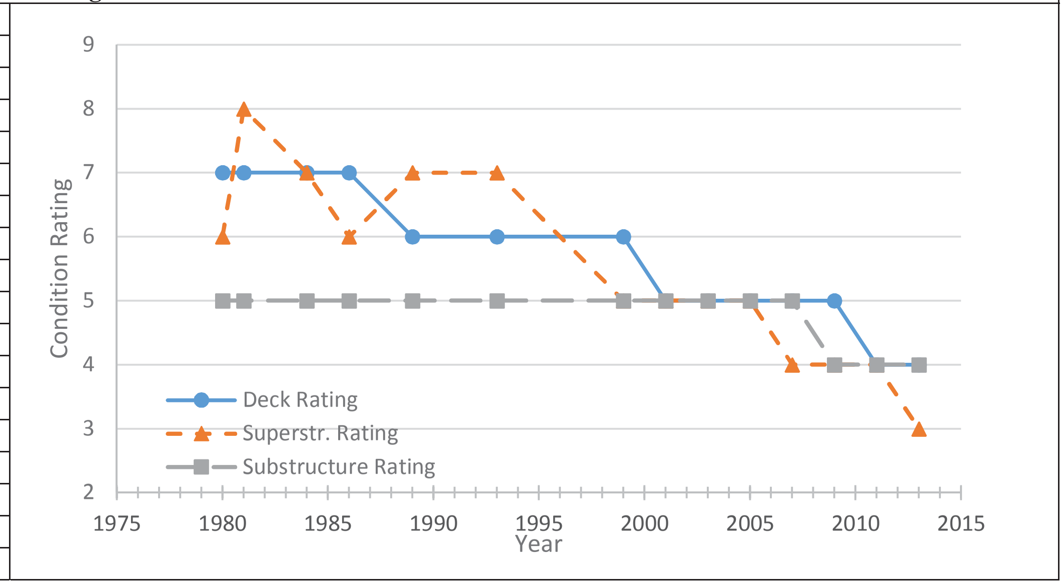


Year	1998	2000	2002	2004	2008	2010	2012
Inspection Interval	72 months	72 months	72 months	72 months	48 months	24 months	24 months



Bridge ID: 07-03-00830 C

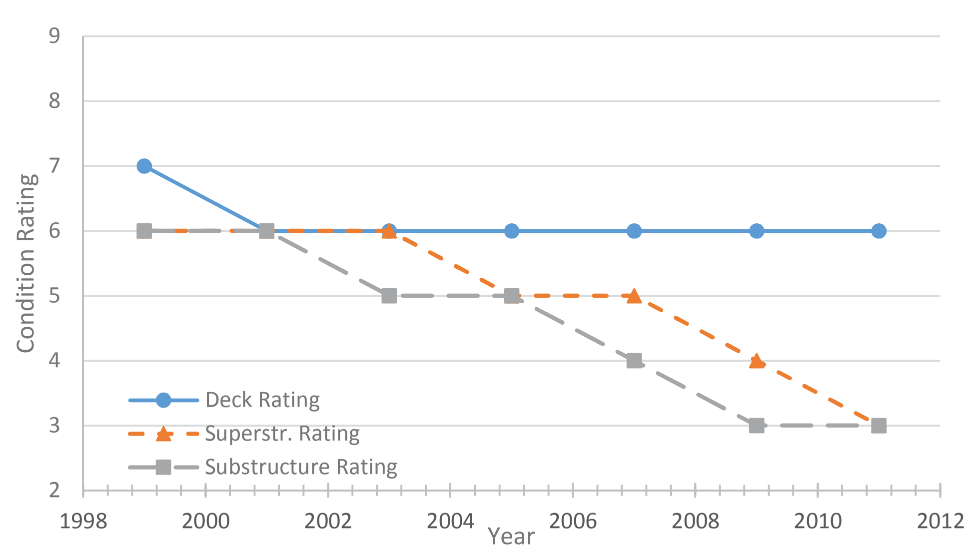
Year Built	1930
No. of Spans	1
Bridge Length (ft.)	65.0
Max. Span Length (ft.)	61.2
Superstructure Type	PCBB
ADT	13,200
ADTT	10%
No. of Lanes	2
Location	Bartholomew County
Environment	Southern District
Deck CR (2013)	4
Superstructure CR (2013)	3
Substructure CR (2013)	4
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew	0
Controlling Damage Mode	Corrosion



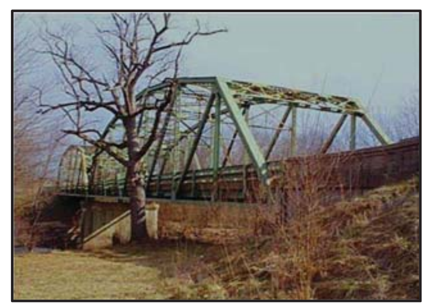
Year	1980	1981	1984	1986	1989	1993	1999	2001	2003	2005
Inspection Interval	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months
Year	2007	2009	2011	2013						
Inspection Interval	24 months	24 months	24 months	24 months						

Bridge ID: 09-73-1994 B

Year Built	1940
No. of Spans	2
Bridge Length (ft.)	304.8
Max. Span Length (ft.)	150.0
Superstructure Type	STT
ADT	3200
ADTT	10%
No. of Lanes	2
Location	Shelby County
Environment	Central District
Deck CR (2013)	6
Superstructure CR (2013)	3
Substructure CR (2013)	3
Facility Under	Waterway
Clearance	14’ 4”
Scour Rating	Stable
Skew	0
Controlling Damage Modes	Fatigue, Corrosion, Fracture

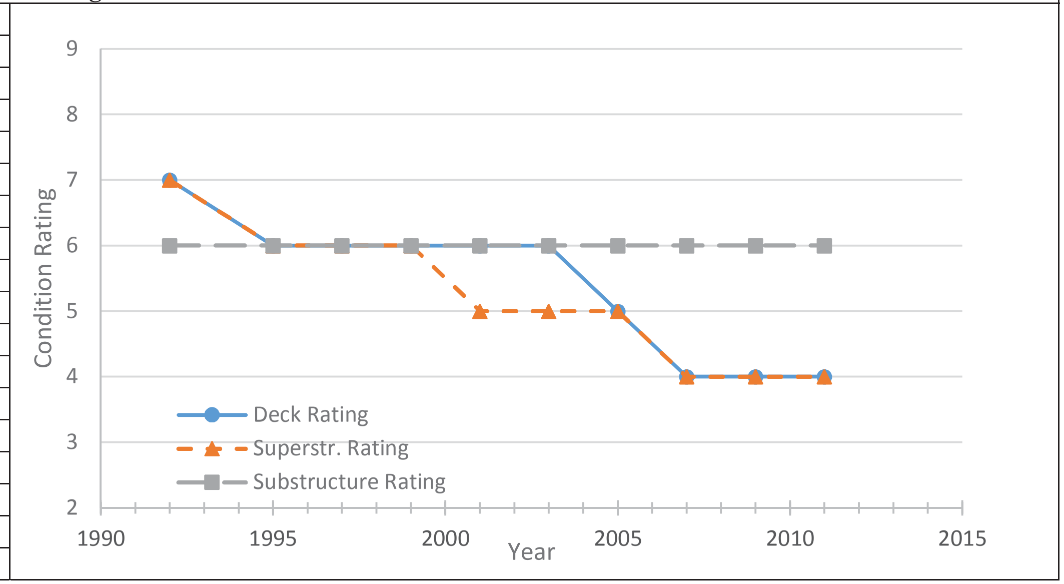


Year	1999	2001	2003	2005	2007	2009	2011
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months

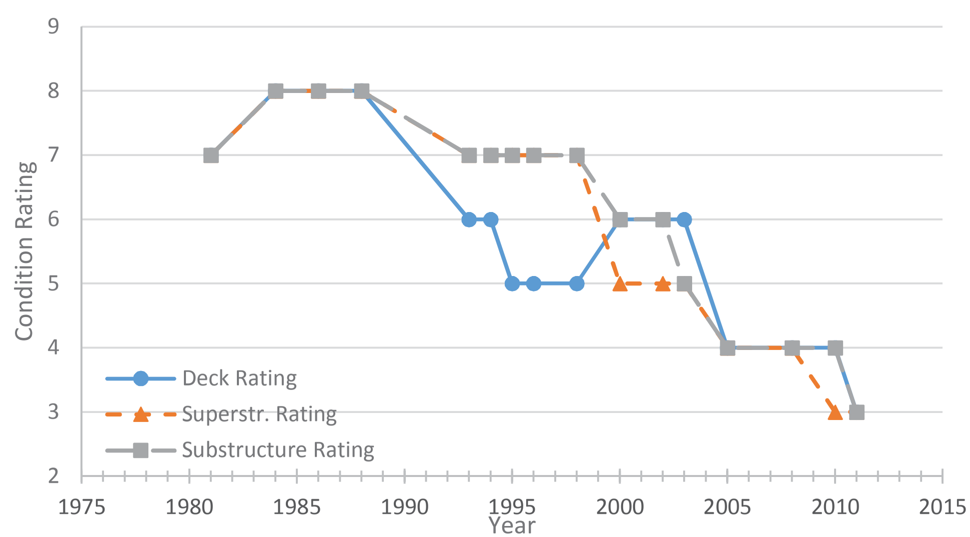


Bridge ID: 18-05-06573 A

Year Built	1976
No. of Spans	3
Bridge Length (ft.)	104.0
Max. Span Length (ft.)	41.0
Superstructure Type	CRCS
ADT	4100
ADTT	10%
No. of Lanes	2
Location	Blackford County
Environment	Northern District
Deck CR (2013)	4
Superstructure CR (2013)	4
Substructure CR (2013)	6
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew	30
Controlling Damage Mode	Corrosion



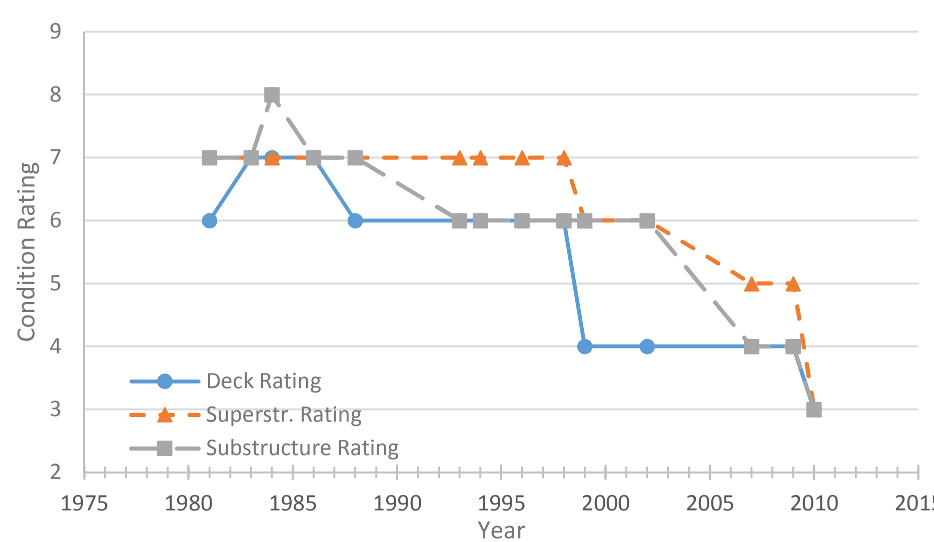
Year	1992	1995	1997	1999	2001	2003	2005	2007	2009	2011
Inspection Interval	72 months	72 months	72 months	72 months	48 months	48 months	24 months	24 months	24 months	24 months

Bridge ID: 24-91-03731 B		
Year Built	1947	
No. of Spans	5	
Bridge Length (ft.)	450.0	
Max. Span Length (ft.)	110.0	
Superstructure Type	RCAOS	
ADT	11300	
ADTT	5%	
No. of Lanes	2	
Location	White County	
Environment	Northern District	
Deck CR (2013)	3	
Superstructure CR (2013)	3	
Substructure CR (2013)	3	
Facility Under	Waterway	
Clearance	N/A	
Scour Rating	Stable	
Skew	0	
Controlling Damage Mode	Corrosion	

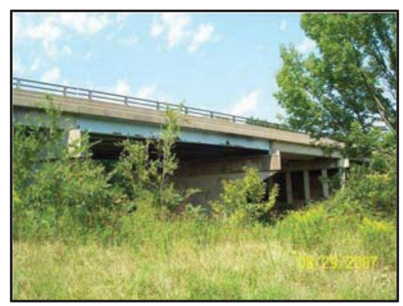
Year	1981	1984	1986	1988	1993	1994	1995	1996	1998	2000
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2002	2003	2005	2008	2010	2011				
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months				

Bridge ID: 25-08-02218 A

Year Built	1963
No. of Spans	1
Bridge Length (ft.)	234.0
Max. Span Length (ft.)	56.0
Superstructure Type	SB
ADT	7300
ADTT	10%
No. of Lanes	2
Location	Carroll County
Environment	Northern District
Deck CR (2013)	3
Superstructure CR (2013)	3
Substructure CR (2013)	3
Facility Under	Railroad
Clearance	22’-06”
Scour Rating	N/A
Skew (Degrees)	37
Controlling Damage Mode	Corrosion

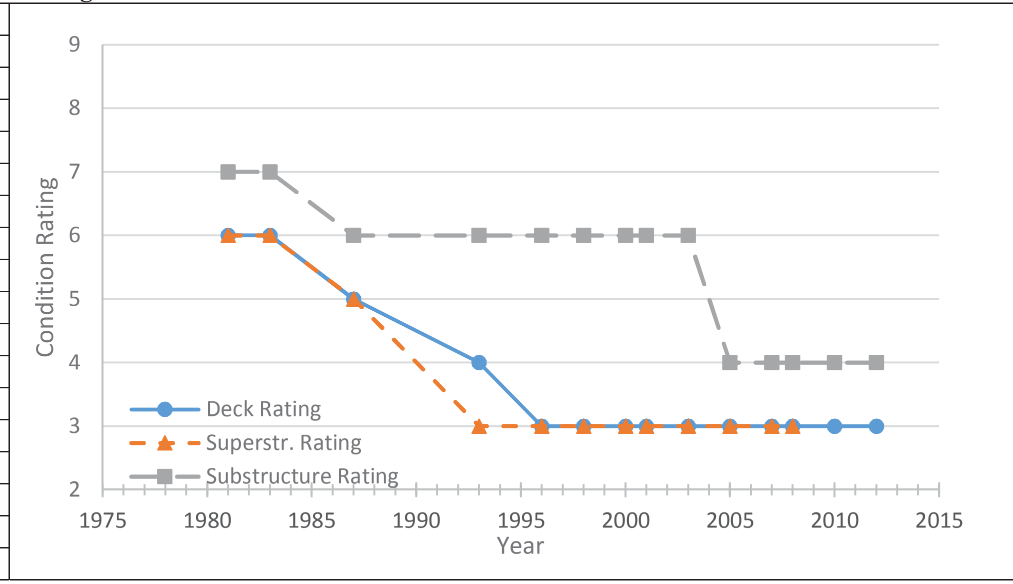


Year	1981	1983	1984	1986	1988	1993	1994	1996	1998
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	1999	2002	2007	2009	2010				
Inspection Interval	24 months	24 months	24 months	24 months	24 months				



Bridge ID: 25-79-2004

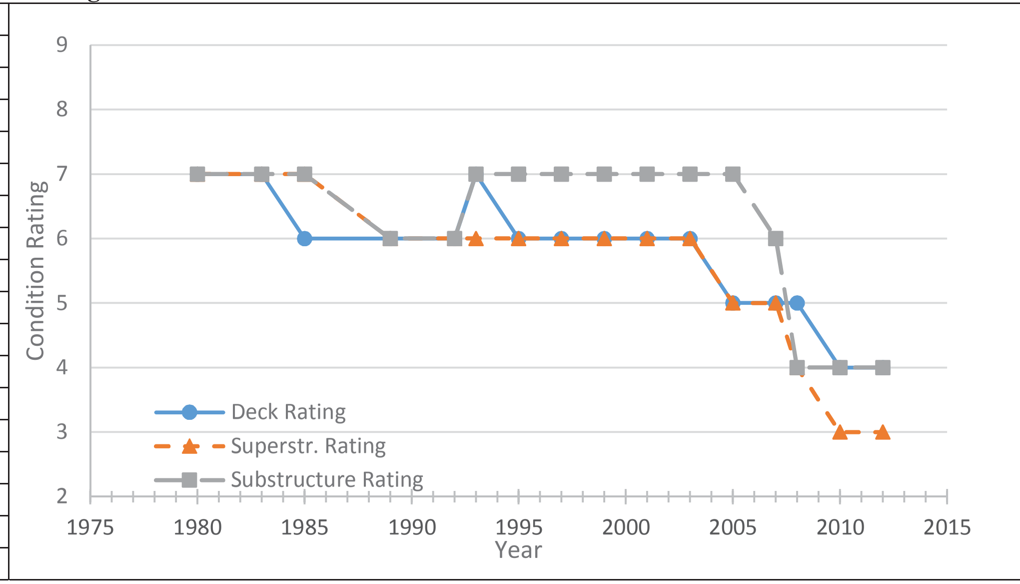
Year Built	1934
No. of Spans	1
Bridge Length (ft.)	46.0
Max. Span Length (ft.)	40.0
Superstructure Type	SB
ADT	Railroad
ADTT	0%
No. of Lanes	0
Location	Tippecanoe County
Environment	Central District
Deck CR (2013)	3
Superstructure CR (2013)	3
Substructure CR (2013)	5
Facility Under	Highway
Clearance	14'-01"
Scour Rating	N/A
Skew (Degrees)	35
Controlling Damage Mode	Fatigue and Fracture



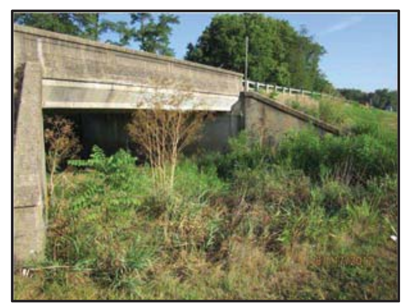
Year	1981	1983	1987	1993	1996	1998	2000	2001	2003
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2005	2007	2008	2010	2012				
Inspection Interval	24 months	24 months	24 months	24 months	24 months				

Bridge ID: 31-36-1773

Year Built	1936
No. of Spans	1
Bridge Length (ft.)	35.0
Max. Span Length (ft.)	32.0
Superstructure Type	RCG
ADT	6700
ADTT	10%
No. of Lanes	2
Location	Jackson County
Environment	Southern District
Deck CR (2013)	4
Superstructure CR (2013)	3
Substructure CR (2013)	4
Facility Under	Waterway
Clearance	N/A
Scour Rating	Scour Critical
Skew (Degrees)	0
Controlling Damage Mode	Corrosion

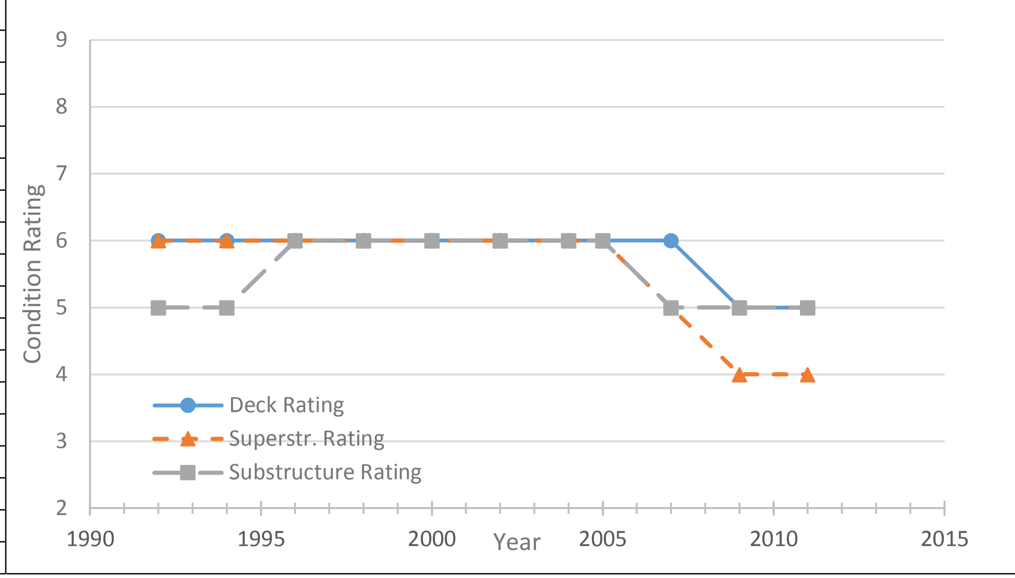


Year	1980	1983	1985	1989	1992	1993	1995	1997	1999	2001
Inspection Interval	72 months	72 months	72 months	72 months	72 months	72 months	72 months	72 months	72 months	72 months
Year	2003	2005	2007	2008	2010	2012				
Inspection Interval	48 months	48 months	24 months	24 months	24 months	24 months				



Bridge ID: 32-06-00339

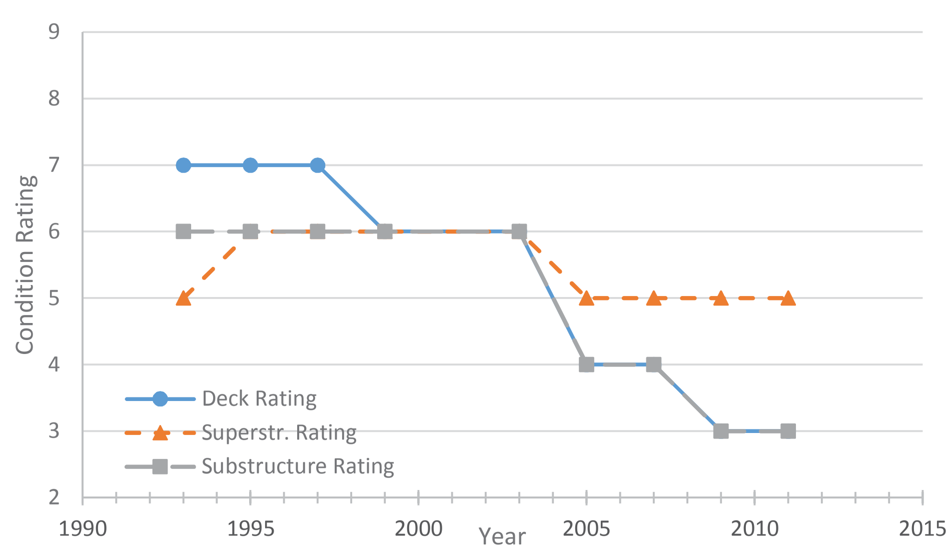
Year Built	1927
No. of Spans	1
Bridge Length (ft.)	30.0
Max. Span Length (ft.)	28.0
Superstructure Type	RCG
ADT	4600
ADTT	10%
No. of Lanes	2
Location	Boone County
Environment	Central District
Deck CR (2013)	5
Superstructure CR (2013)	4
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	N/A
Scour Rating	Scour Critical
Skew (Degrees)	30
Controlling Damage Mode	Corrosion



Year	1992	1994	1996	1998	2000	2002	2004	2005	2007	2009	2011
Inspection Interval	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	24 months	24 months

Bridge ID: 39-06-4115 C

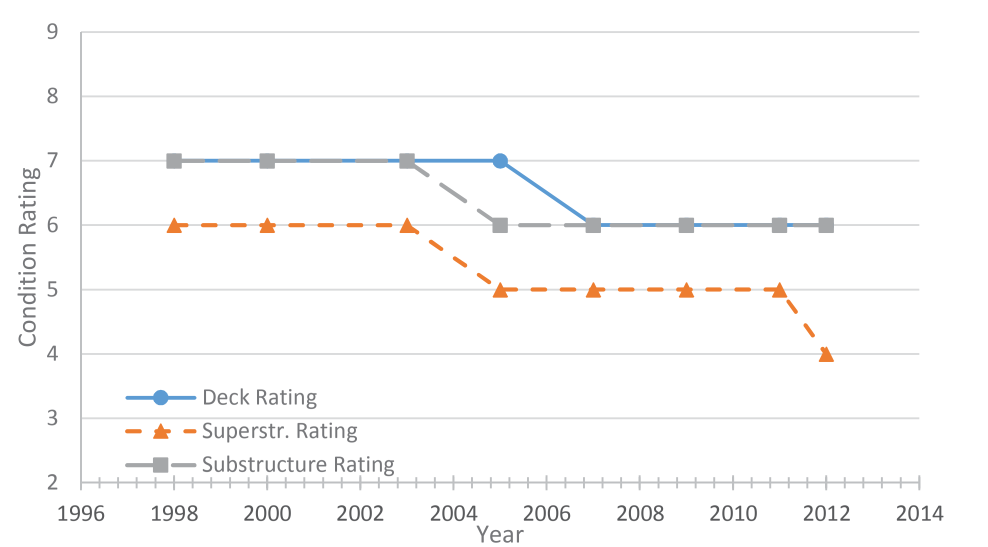
Year Built	1958
No. of Spans	4
Bridge Length (ft.)	212.0
Max. Span Length (ft.)	64.0
Superstructure Type	CSB
ADT	27700
ADTT	11%
No. of Lanes	4
Location	Boone County
Environment	Central District
Deck CR (2013)	3
Superstructure CR (2013)	5
Substructure CR (2013)	3
Facility Under	Highway
Clearance	14’-07”
Scour Rating	N/A
Skew (Degrees)	18
Controlling Damage Mode	Fatigue



Year	1993	1995	1997	1999	2003	2005	2007	2009	2011
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months

Bridge ID: 40-30-04589 B

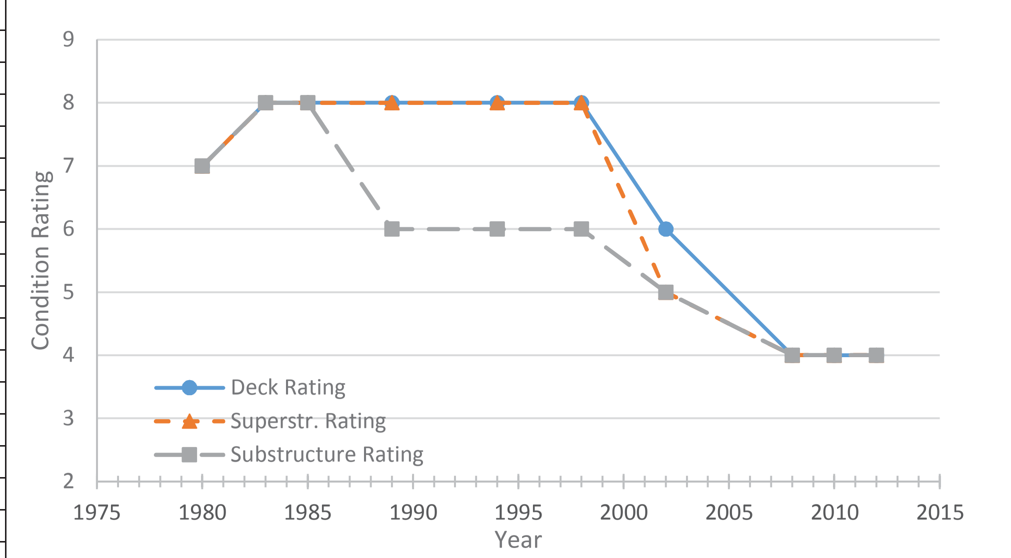
Year Built	1959
No. of Spans	3
Bridge Length (ft.)	178.3
Max. Span Length (ft.)	69.0
Superstructure Type	CRCG
ADT	25300
ADTT	10%
No. of Lanes	4
Location	Hancock County
Environment	Central District
Deck CR (2013)	6
Superstructure CR (2013)	4
Substructure CR (2013)	6
Facility Under	Waterway
Clearance	N/A
Scour Rating	Fixed
Skew (Degrees)	0
Controlling Damage Mode	Corrosion



Year	1998	2000	2003	2005	2007	2009	2011	2012
Inspection Interval	48 months	48 months	48 months	24 months	24 months	24 months	24 months	24 months

Bridge ID: 41-45-02039 A

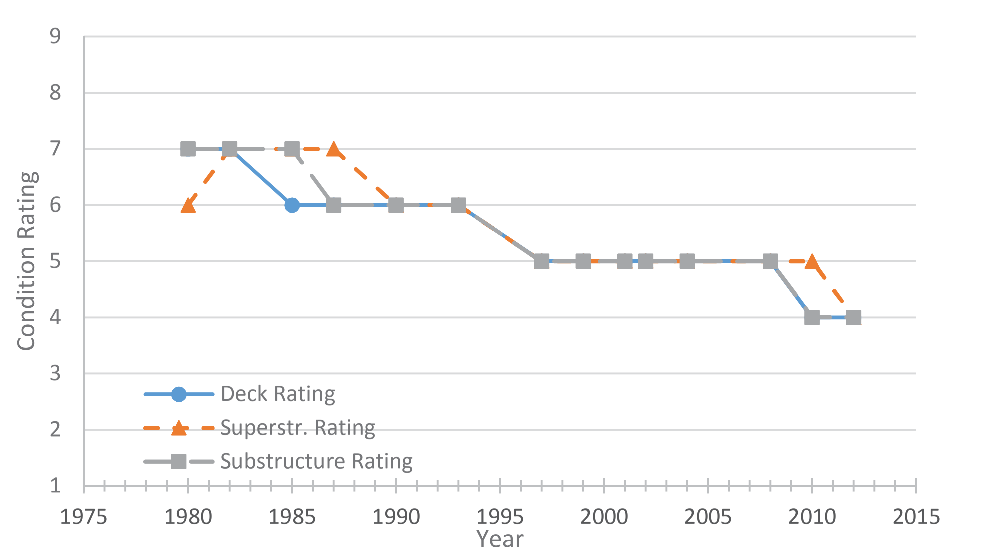
Year Built	1936
No. of Spans	12
Bridge Length (ft.)	481.0
Max. Span Length (ft.)	68.1
Superstructure Type	SB
ADT	34200
ADTT	5%
No. of Lanes	4
Location	Lake County
Environment	Central District
Deck CR (2013)	4
Superstructure CR (2013)	4
Substructure CR (2013)	4
Facility Under	Other
Clearance	N/A
Scour Rating	N/A
Skew (Degrees)	39
Controlling Damage Mode	Corrosion



Year	1980	1983	1985	1989	1994	1998	2002	2008	2010	2012
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months

Bridge ID: 45-28-03529

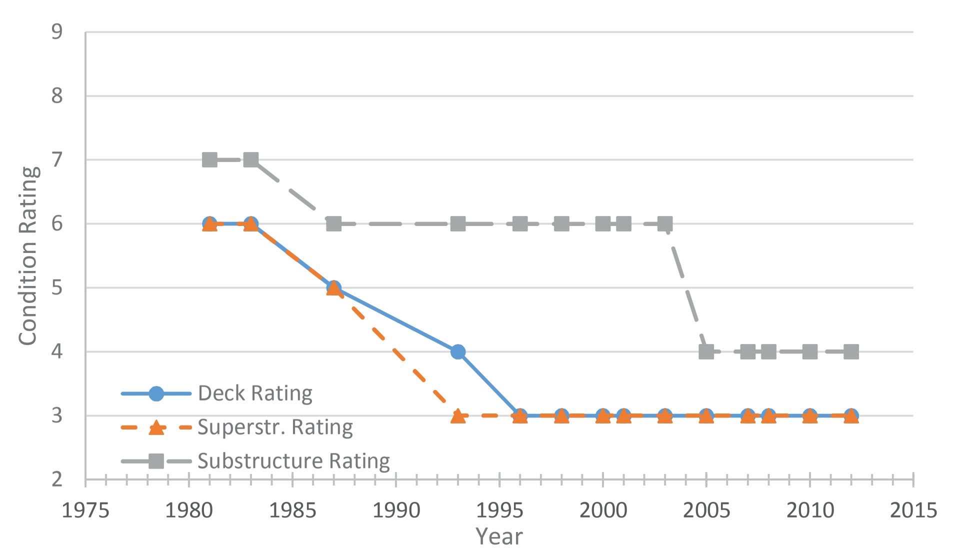
Year Built	1946
No. of Spans	3
Bridge Length (ft.)	62.0
Max. Span Length (ft.)	24.0
Superstructure Type	CRCS
ADT	2600
ADTT	10%
No. of Lanes	2
Location	Greene County
Environment	Southern District
Deck CR (2013)	4
Superstructure CR (2013)	4
Substructure CR (2013)	4
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	30
Controlling Damage Mode	Corrosion



Year	1980	1982	1985	1987	1990	1993	1997	1999	2001
Inspection Interval	72 months	72 months	72 months	72 months	48 months	48 months	48 months	48 months	48 months
Year	2002	2004	2008	2010	2012				
Inspection Interval	48 months	48 months	48 months	24 months	24 months				

Bridge ID: 46-11-01313 A

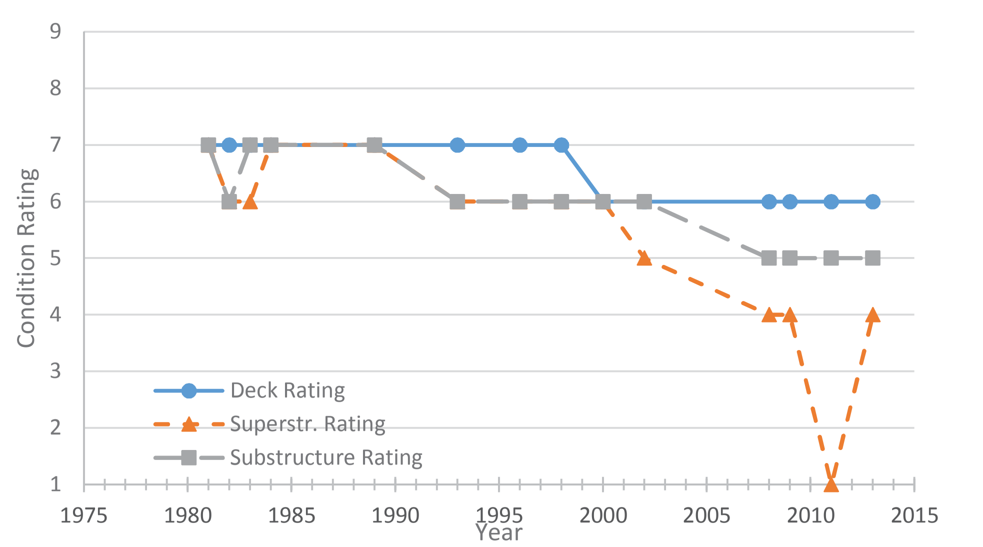
Year Built	1932
No. of Spans	1
Bridge Length (ft.)	100.0
Max. Span Length (ft.)	96.0
Superstructure Type	SPT
ADT	Waterway
ADTT	3400
No. of Lanes	2
Location	Clay County
Environment	Central District
Deck CR (2013)	4
Superstructure CR (2013)	3
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	0
Controlling Damage Mode	Fatigue



Year	1981	1982	1983	1984	1991	1993	1995	1997	1999
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2001	2003	2005	2007	2008	2010	2011	2012	
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	

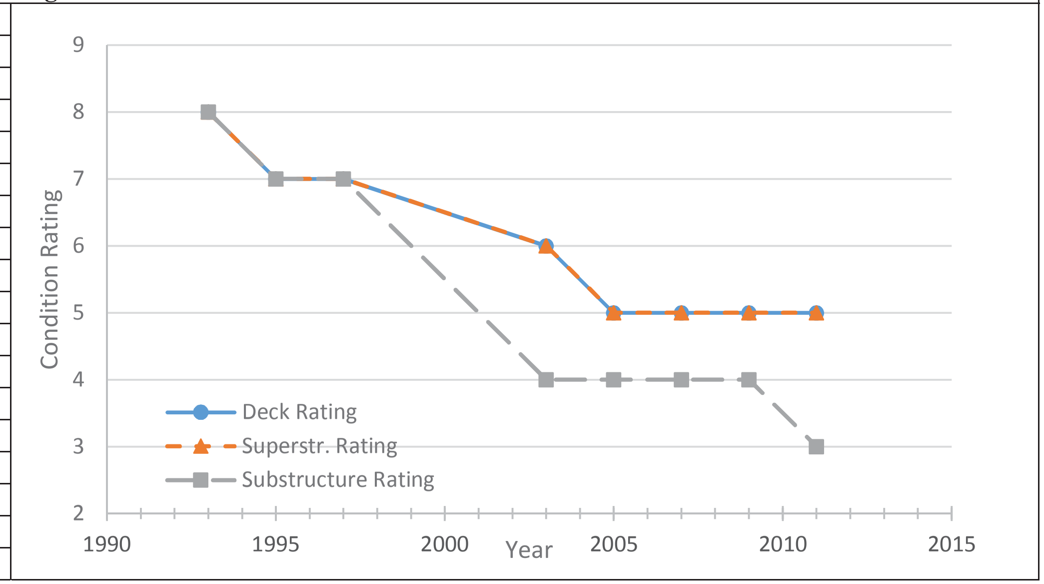
Bridge ID: 46-11-01316 B

Year Built	1934
No. of Spans	2
Bridge Length (ft.)	402.3
Max. Span Length (ft.)	198.0
Superstructure Type	STT
ADT	2400
ADTT	3%
No. of Lanes	2
Location	Clay County
Environment	Central District
Deck CR (2013)	6
Superstructure CR (2013)	4
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	14’-08”
Scour Rating	Stable
Skew (Degrees)	0
Controlling Damage Mode	Fatigue



Year	1981	1982	1983	1984	1989	1993	1996	1998	2000
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2002	2008	2009	2011					
Inspection Interval	24 months	24 months	24 months	24 months					

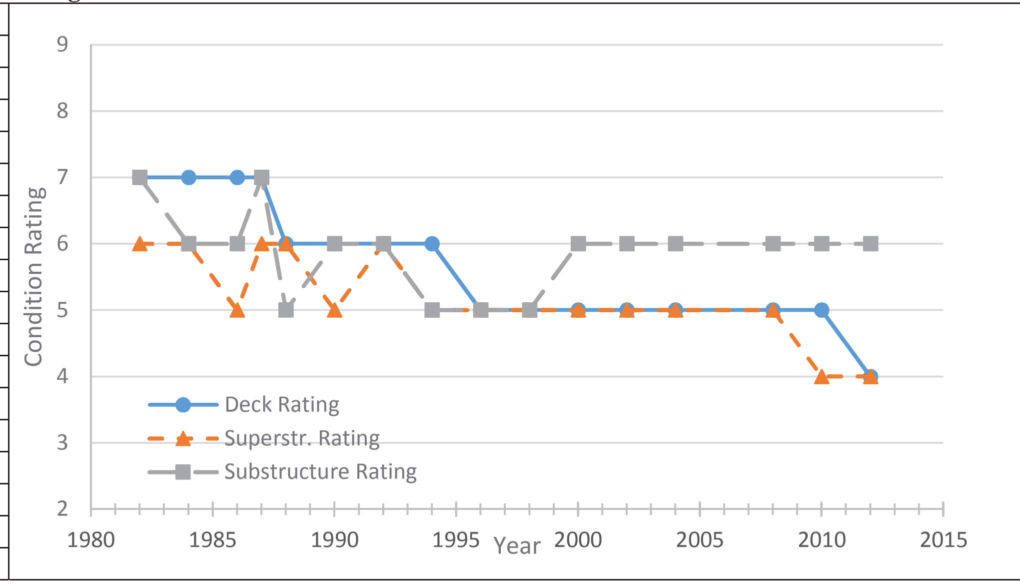
Bridge ID: 51-45-05941 ASBL	
Year Built	1968
No. of Spans	4
Bridge Length (ft.)	173.0
Max. Span Length (ft.)	42.5
Superstructure Type	CPCIB
ADT	15600
ADTT	5%
No. of Lanes	2
Location	Lake County
Environment	Northern District
Deck CR (2013)	5
Superstructure CR (2013)	5
Substructure CR (2013)	3
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	20
Controlling Damage Mode	Corrosion



Year	1993	1995	1997	2003	2005	2007	2009	2011
Inspection Interval	72 months	72 months	48 months	24 months	24 months	24 months	24 months	24 months

Bridge ID: 54-77-00343 A

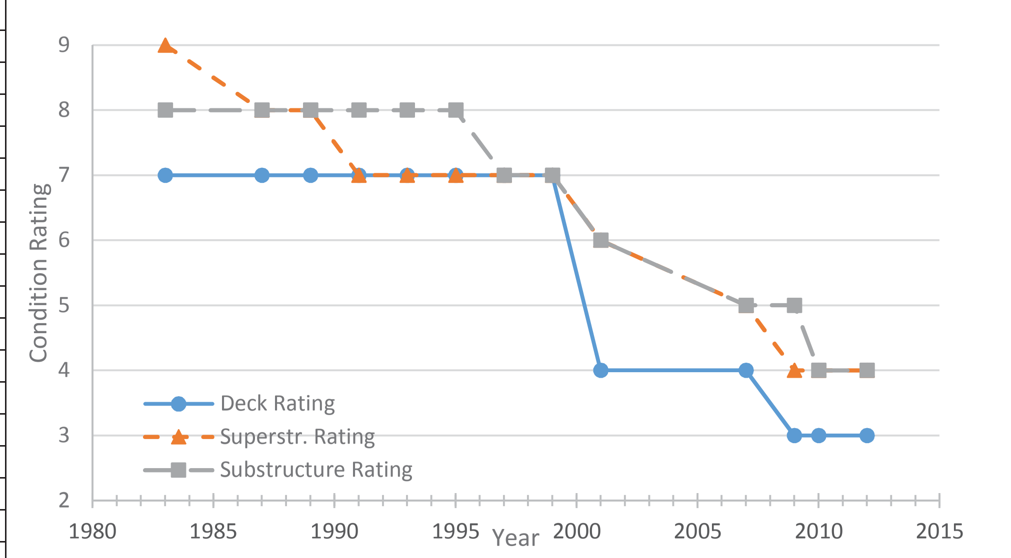
Year Built	1924
No. of Spans	3
Bridge Length (ft.)	85.0
Max. Span Length (ft.)	36.0
Superstructure Type	PCB
ADT	5500
ADTT	10%
No. of Lanes	2
Location	Sullivan County
Environment	Southern District
Deck CR (2013)	4
Superstructure CR (2013)	4
Substructure CR (2013)	6
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	0
Controlling Damage Mode	Fatigue



Year	1982	1984	1986	1987	1988	1990	1992	1994	1996	1998
Inspection Interval	72 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months
Year	2000	2002	2004	2008	2010	2012				
Inspection Interval	48 months	48 months	48 months	48 months	24 months	24 months				

Bridge ID: 55-45-06258 B

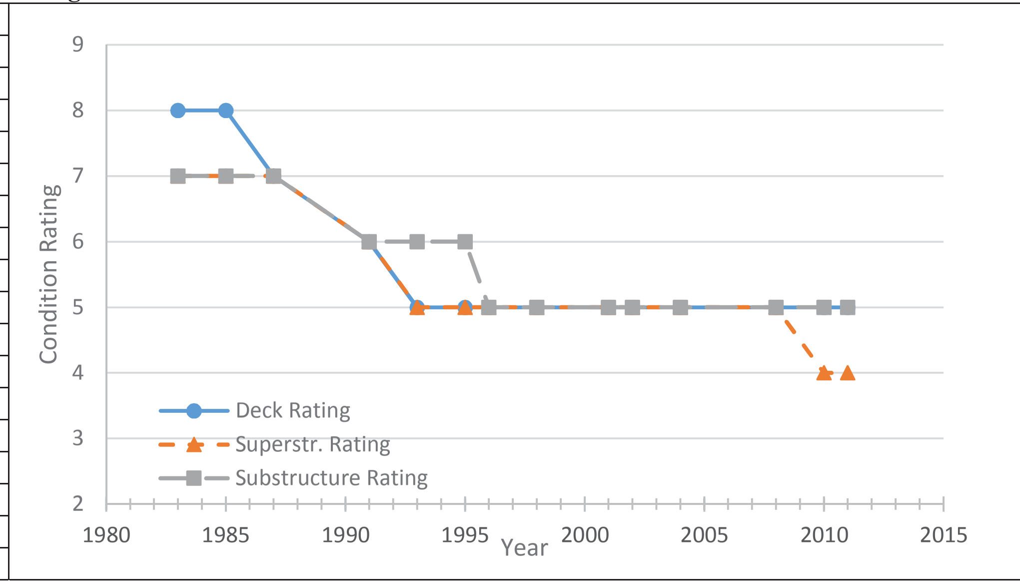
Year Built	1964
No. of Spans	3
Bridge Length (ft.)	100.0
Max. Span Length (ft.)	42.0
Superstructure Type	PCBB
ADT	1700
ADTT	10%
No. of Lanes	2
Location	Lake County
Environment	Northern District
Deck CR (2013)	3
Superstructure CR (2013)	4
Substructure CR (2013)	4
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	19
Controlling Damage Mode	Corrosion



Year	1983	1987	1989	1991	1993	1995	1997	1999	2001
Inspection Interval	72 months	72 months	72 months	72 months	72 months	72 months	72 months	72 months	48 months
Year	2007	2009	2010	2012					
Inspection Interval	48 months	24 months	24 months	24 months					

Bridge ID: 57-28-00341 C

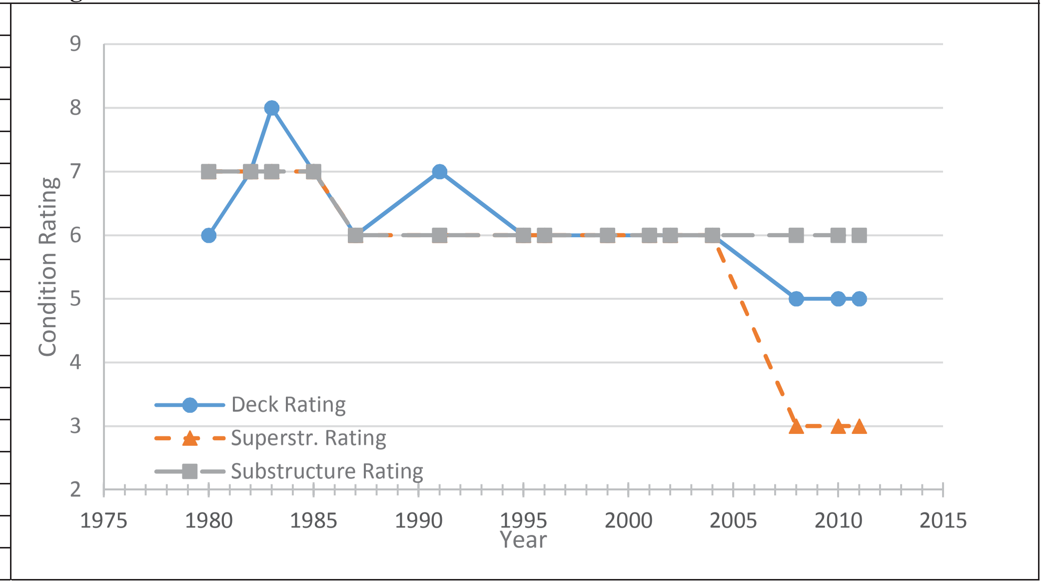
Year Built	1940
No. of Spans	10
Bridge Length (ft.)	1002.0
Max. Span Length (ft.)	200.0
Superstucture Type	STT
ADT	3400
ADTT	10%
No. of Lanes	2
Location	Greene County
Environment	Southern District
Deck CR (2013)	5
Superstructure CR (2013)	4
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	14’-8”
Scour Rating	Stable
Skew (Degrees)	0
Controlling Damage Mode	Fatigue



Year	1983	1985	1987	1991	1993	1995	1996	1998	2001
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2002	2004	2008	2010	2011				
Inspection Interval	24 months	24 months	24 months	24 months	24 months				

Bridge ID: 57-28-03042 D

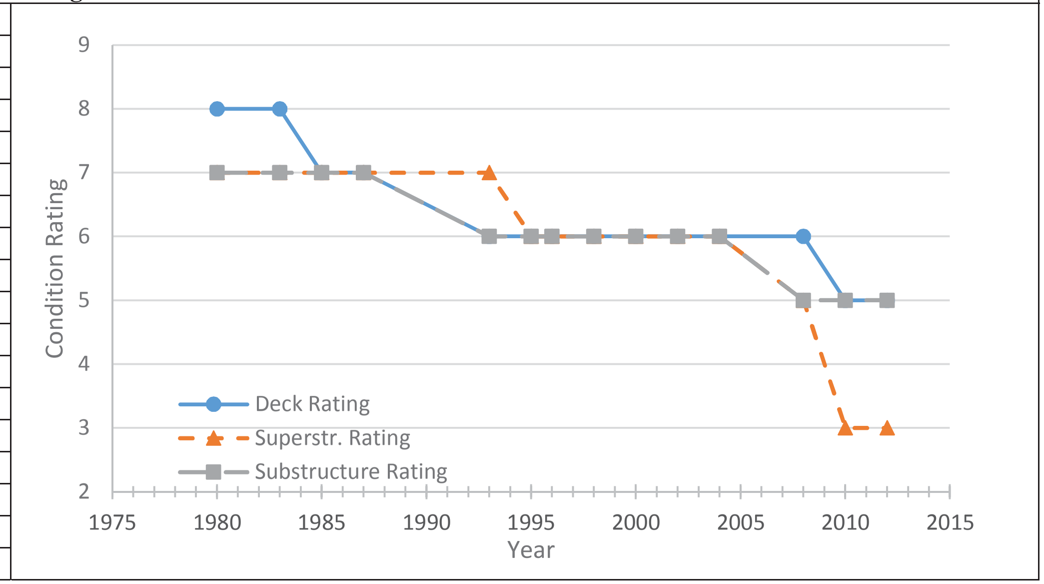
Year Built	1940
No. of Spans	4
Bridge Length (ft.)	410.0
Max. Span Length (ft.)	150.0
Superstructure Type	STT
ADT	3400
ADTT	10%
No. of Lanes	1
Location	Greene County
Environment	Southern District
Deck CR (2013)	5
Superstructure CR (2013)	3
Substructure CR (2013)	6
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	0
Controlling Damage Mode	Fatigue



Year	1980	1982	1983	1985	1987	1991	1995	1996	1999
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2001	2002	2004	2008	2010	2012			
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months			

Bridge ID: 58-36-03059 C

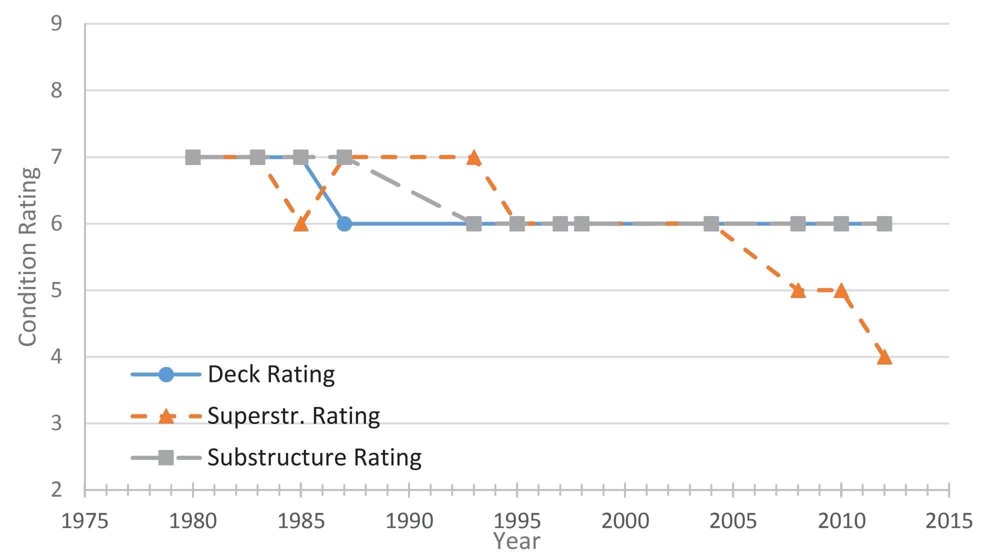
Year Built	1961
No. of Spans	1
Bridge Length (ft.)	58.0
Max. Span Length (ft.)	56.0
Superstructure Type	PCBB
ADT	1100
ADTT	10%
No. of Lanes	2
Location	Jackson County
Environment	Southern District
Deck CR (2013)	5
Superstructure CR (2013)	3
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	N/A
Scour Rating	Fixed
Skew (Degrees)	28
Controlling Damage Mode	Corrosion



Year	1980	1983	1985	1987	1993	1995	1996	1998	2000	2002
Inspection Interval	72 months	72 months	72 months	72 months	72 months	48 months	48 months	48 months	48 months	48 months
Year	2004	2008	2010	2012						
Inspection Interval	48 months	48 months	48 months	24 months						

Bridge ID: 58-36-05871

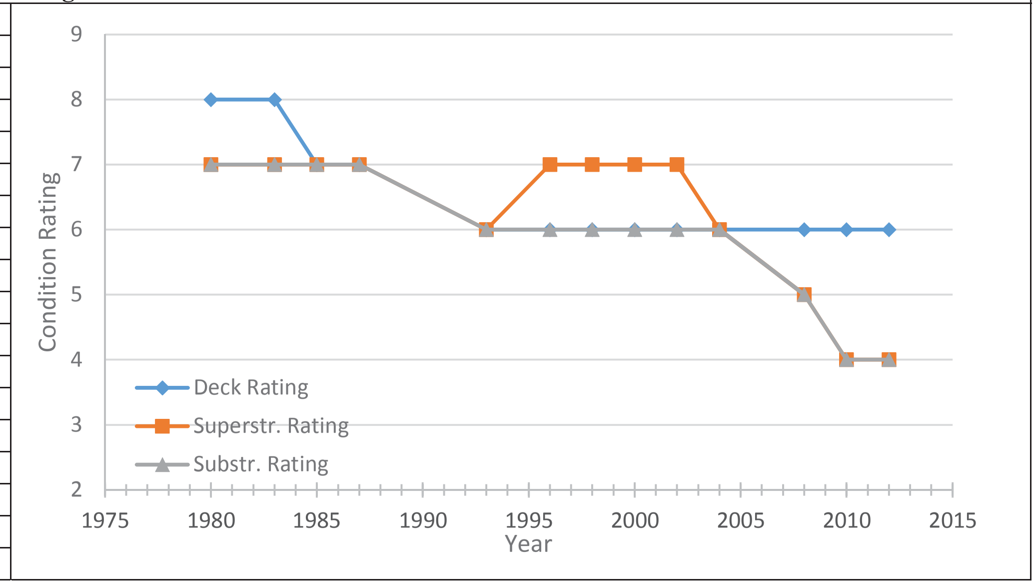
Year Built	1967
No. of Spans	1
Bridge Length (ft.)	38.0
Max. Span Length (ft.)	36.0
Superstructure Type	PCB
ADT	1300
ADTT	10%
No. of Lanes	2
Location	Jackson County
Environment	Southern District
Deck CR (2013)	6
Superstructure CR (2013)	4
Substructure CR (2013)	6
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	0
Controlling Damage Mode	Corrosion



Year	1980	1983	1985	1987	1993	1995	1997	1998
Inspection Interval	72 months	72 months	72 months	72 months	72 months	72 months	72 months	48 months
Year	2004	2008	2010	2012				
Inspection Interval	48 months	48 months	48 months	24 months				

Bridge ID: 58-36-06415 C

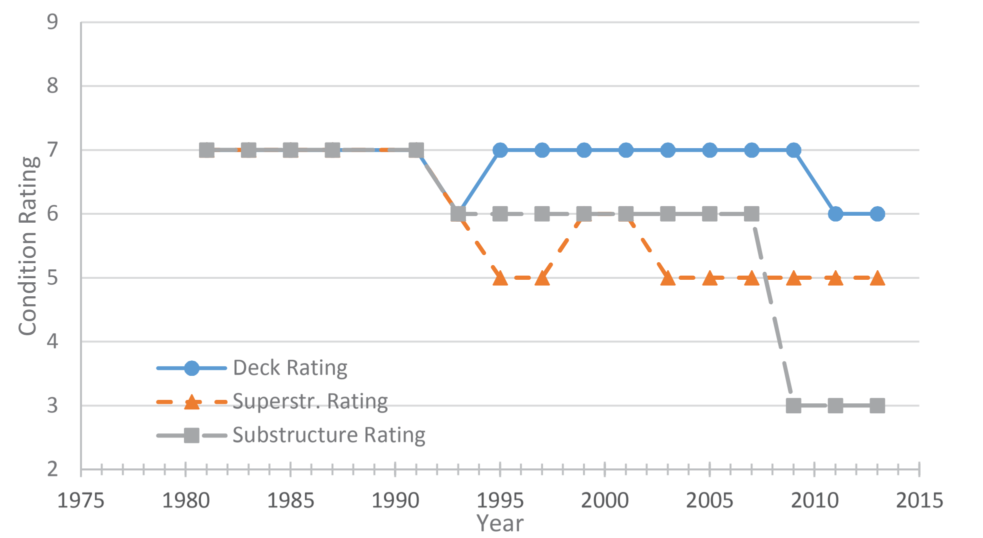
Year Built	1925
No. of Spans	1
Bridge Length (ft.)	26.0
Max. Span Length (ft.)	23.1
Superstructure Type	PCBB
ADT	2300
ADTT	10%
No. of Lanes	2
Location	Jackson County
Environment	Southern District
Deck CR (2013)	6
Superstructure CR (2013)	4
Substructure CR (2013)	4
Facility Under	Waterway
Clearance	N/A
Scour Rating	Fixed
Skew (Degrees)	0
Controlling Damage Mode	Corrosion



Year	1980	1983	1985	1987	1993	1996	1998	2000	2002
Inspection Interval	72 months	72 months	72 months	72 months	72 months	72 months	48 months	48 months	48 months
Year	2004	2008	2010	2012					
Inspection Interval	48 months	48 months	24 months	24 months					

Bridge ID: 66-62-05756 B

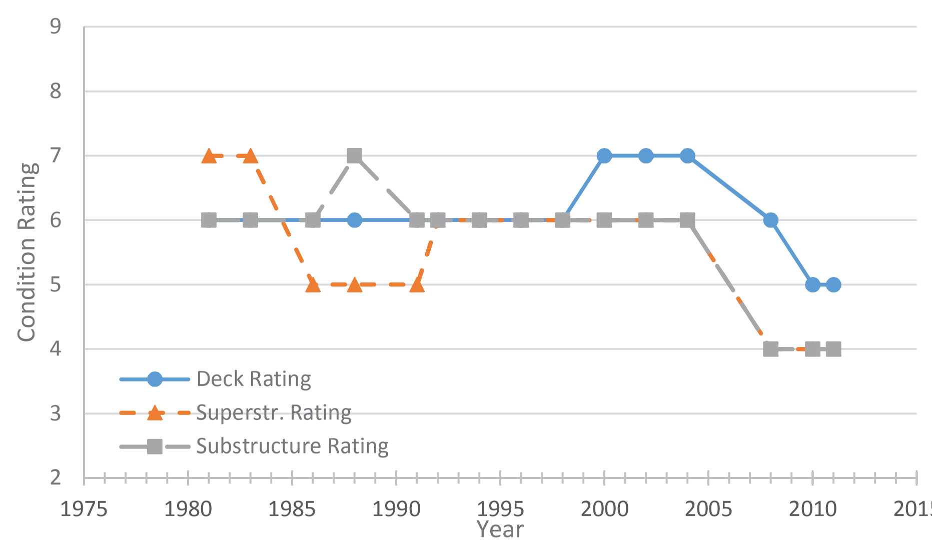
Year Built	1969
No. of Spans	5
Bridge Length (ft.)	257.0
Max. Span Length (ft.)	48.8
Superstructure Type	CPCBB
ADT	1800
ADTT	10%
No. of Lanes	2
Location	Perry County
Environment	Southern District
Deck CR (2013)	6
Superstructure CR (2013)	5
Substructure CR (2013)	3
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	0
Controlling Damage Mode	Corrosion



Year	1981	1983	1985	1987	1991	1993	1995	1997	1999	2001
Inspection Interval	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months
Year	2003	2005	2007	2009	2011	2013				
Inspection Interval	48 months	48 months	24 months	24 months	24 months	24 months				

Bridge ID: 75-06-06480 A

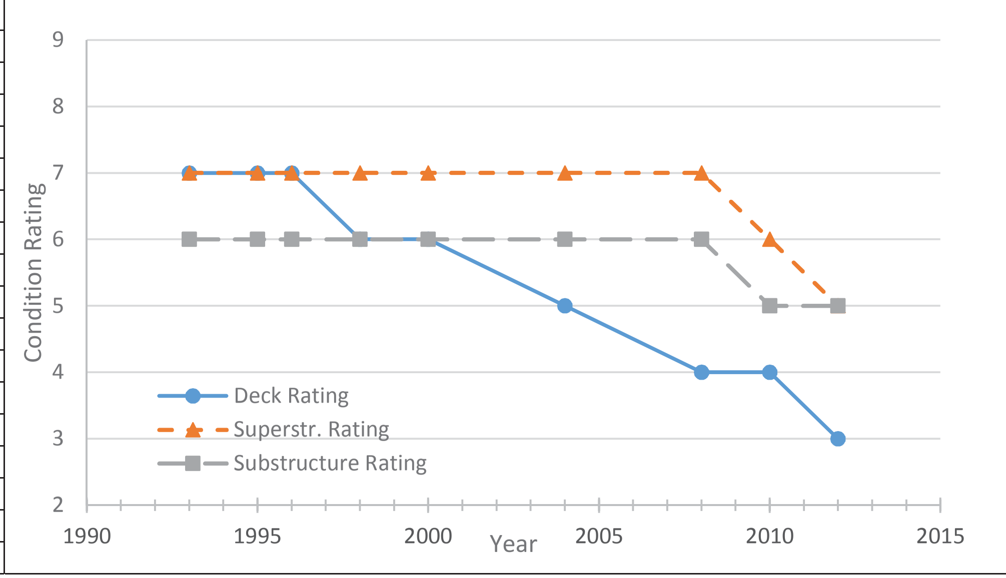
Year Built	1936
No. of Spans	1
Bridge Length (ft.)	34.0
Max. Span Length (ft.)	32.0
Superstructure Type	PCBB
ADT	2100
ADTT	7%
No. of Lanes	2
Location	Boone County
Environment	Central District
Deck CR (2013)	5
Superstructure CR (2013)	4
Substructure CR (2013)	4
Facility Under	Waterway
Clearance	N/A
Scour Rating	Fixed
Skew (Degrees)	48
Controlling Damage Mode	Strand Fracture



Year	1981	1983	1986	1988	1991	1992	1994	1996	1998	2000
Inspection Interval	72 months	72 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months
Year	2002	2004	2008	2010	2011					
Inspection Interval	48 months	48 months	24 months	24 months	24 months					

Bridge ID: 135-07-04053 A

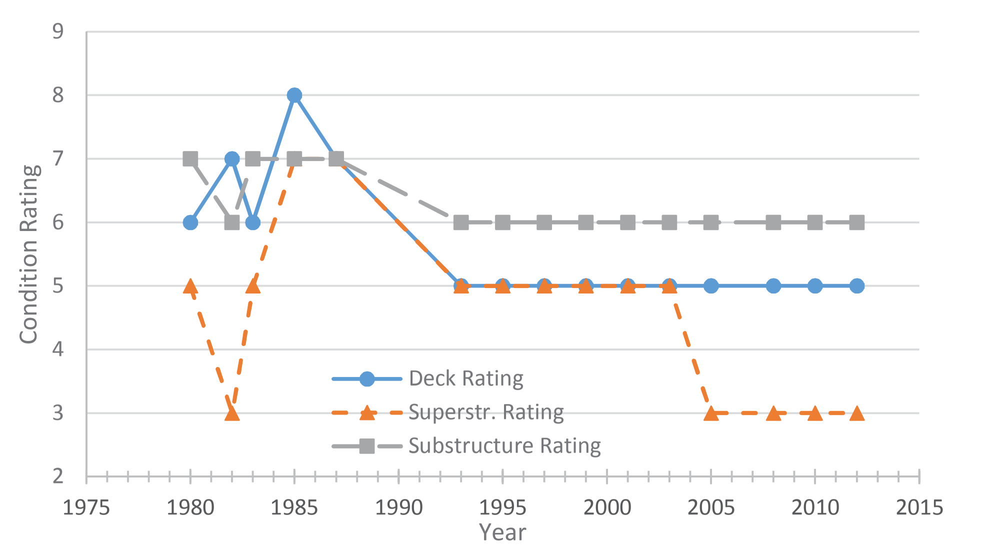
Year Built	1961
No. of Spans	3
Bridge Length (ft.)	125.0
Max. Span Length (ft.)	38.0
Superstructure Type	RCG
ADT	2000
ADTT	10%
No. of Lanes	2
Location	Brown County
Environment	Southern District
Deck CR (2013)	3
Superstructure CR (2013)	5
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	10
Controlling Damage Mode	Corrosion



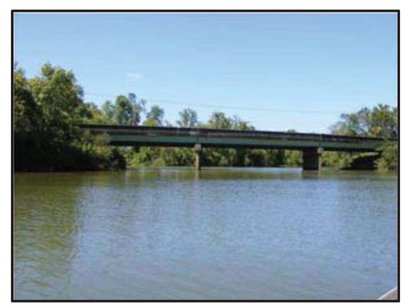
Year	1993	1995	1996	1998	2000	2004	2008	2010	2012
Inspection Interval	72 months	72 months	48 months	48 months	48 months	48 months	24 months	24 months	48 months

Bridge ID: 166-62-01995 A

Year Built	1938
No. of Spans	3
Bridge Length (ft.)	207.0
Max. Span Length (ft.)	66.8
Superstructure Type	SB
ADT	100
ADTT	10%
No. of Lanes	2
Location	Perry County
Environment	Southern District
Deck CR (2013)	5
Superstructure CR (2013)	3
Substructure CR (2013)	6
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	25
Controlling Damage Mode	Fatigue

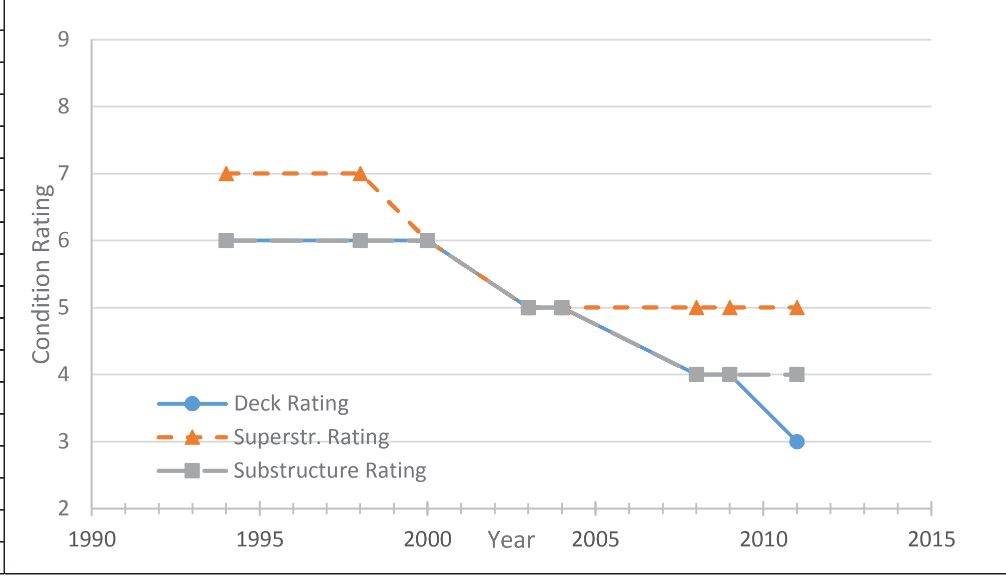


Year	1980	1982	1983	1985	1987	1993	1995	1997	1999	2001
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2003	2005	2008	2010	2012					
Inspection Interval	24 months	24 months	24 months	24 months	24 months					



Bridge ID: 203-72-03078 A

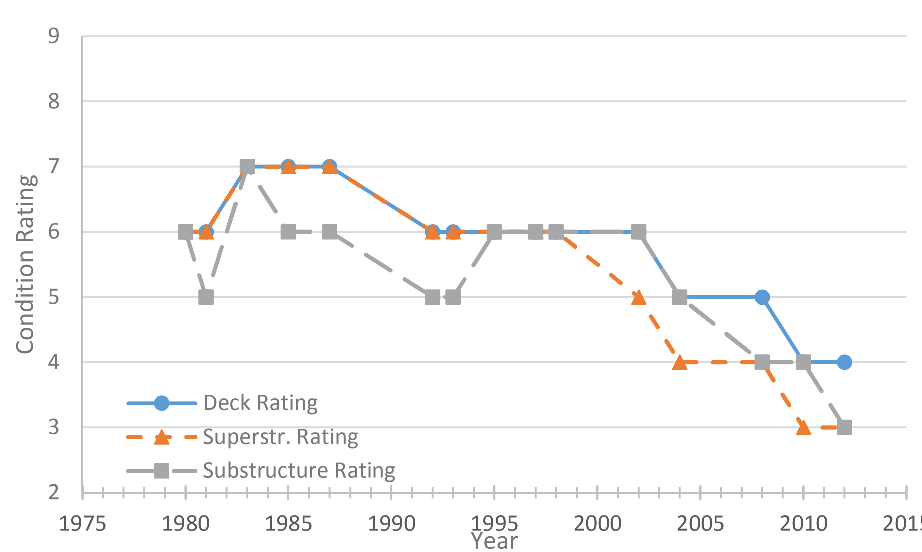
Year Built	1963
No. of Spans	3
Bridge Length (ft.)	155.0
Max. Span Length (ft.)	60.0
Superstructure Type	PCIB
ADT	600
ADTT	10%
No. of Lanes	2
Location	Scott County
Environment	Southern District
Deck CR (2013)	3
Superstructure CR (2013)	5
Substructure CR (2013)	4
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	30
Controlling Damage Mode	Corrosion



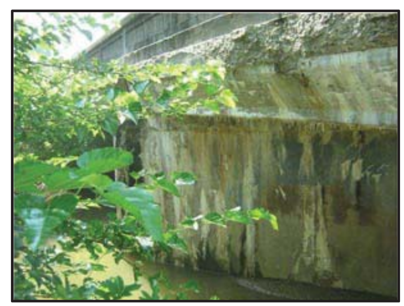
Year	1994	1998	2000	2003	2004	2008	2009	2011
Inspection Interval	72 months	48 months	48 months	48 months	48 months	24 months	24 months	24 months

Bridge ID: 252-55-01968

Year Built	1937
No. of Spans	2
Bridge Length (ft.)	66.0
Max. Span Length (ft.)	28.0
Superstructure Type	RCG
ADT	5100
ADTT	5%
No. of Lanes	2
Location	Morgan County
Environment	Southern District
Deck CR (2013)	4
Superstructure CR (2013)	3
Substructure CR (2013)	3
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	56
Controlling Damage Mode	Corrosion

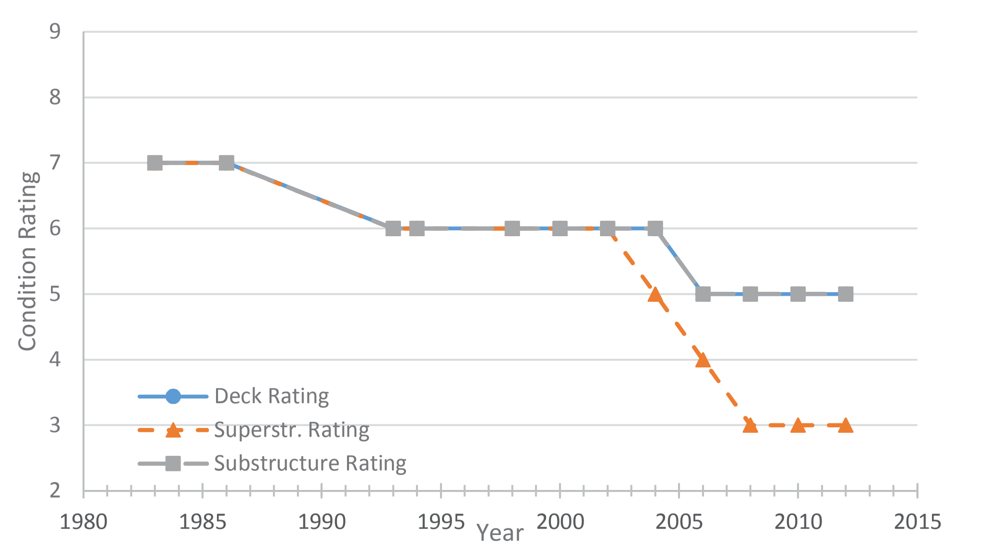


Year	1980	1981	1983	1985	1987	1992	1993	1995	1997	1998
Inspection Interval	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months	48 months
Year	2002	2004	2008	2010	2012					
Inspection Interval	24 months	24 months	24 months	24 months	24 months					



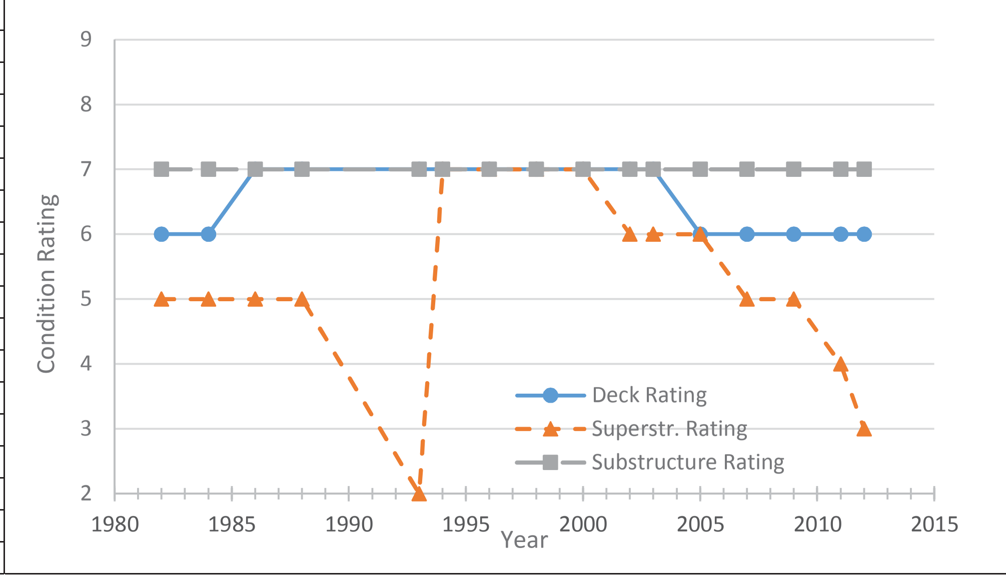
Bridge ID: 356-72-06433 B

Year Built	1920
No. of Spans	1
Bridge Length (ft.)	72.0
Max. Span Length (ft.)	71.3
Superstructure Type	PCBB
ADT	500
ADTT	10%
No. of Lanes	2
Location	Scott County
Environment	Southern District
Deck CR (2013)	5
Superstructure CR (2013)	3
Substructure CR (2013)	5
Facility Under	Waterway
Clearance	N/A
Scour Rating	Stable
Skew (Degrees)	0
Controlling Damage Mode	Corrosion



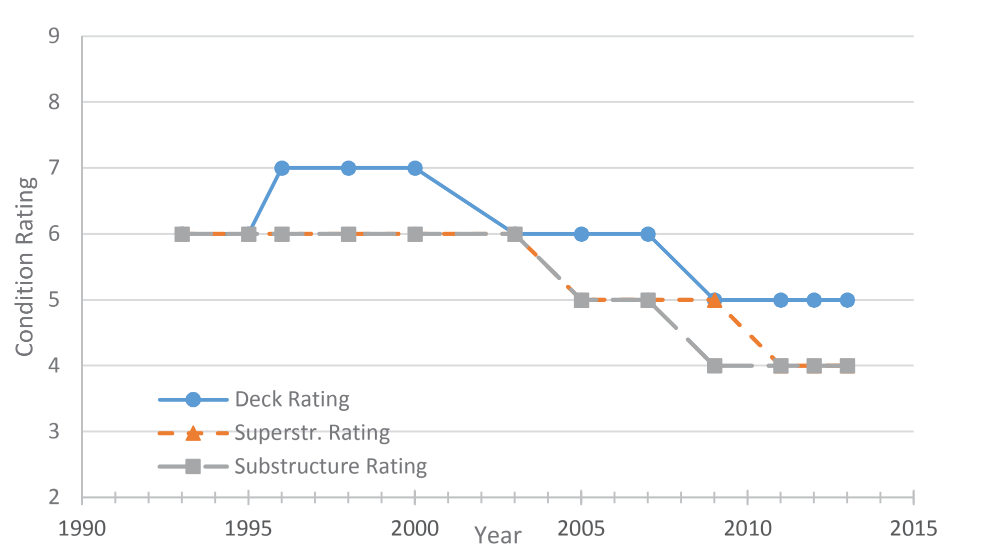
Year	1983	1986	1993	1994	1998	2000	2002	2004
Inspection Interval	72 months	72 months	48 months	48 months	48 months	48 months	48 months	24 months
Year	2006	2008	2010					
Inspection Interval	24 months	24 months	24 months					

Bridge ID: I65-14-04218 B	
Year Built	1958
No. of Spans	4
Bridge Length (ft.)	232.0
Max. Span Length (ft.)	71.3
Superstructure Type	CSB
ADT	400
ADTT	10%
No. of Lanes	2
Location	Clark County
Environment	Southern District
Deck CR (2013)	6
Superstructure CR (2013)	3
Substructure CR (2013)	7
Facility Under	Highway
Clearance	14’-07”
Scour Rating	N/A
Skew (Degrees)	24
Controlling Damage Mode	Fatigue



Year	1982	1984	1986	1988	1993	1994	1996	1998	2000
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2002	2003	2005	2007	2009	2011	2012		
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months		

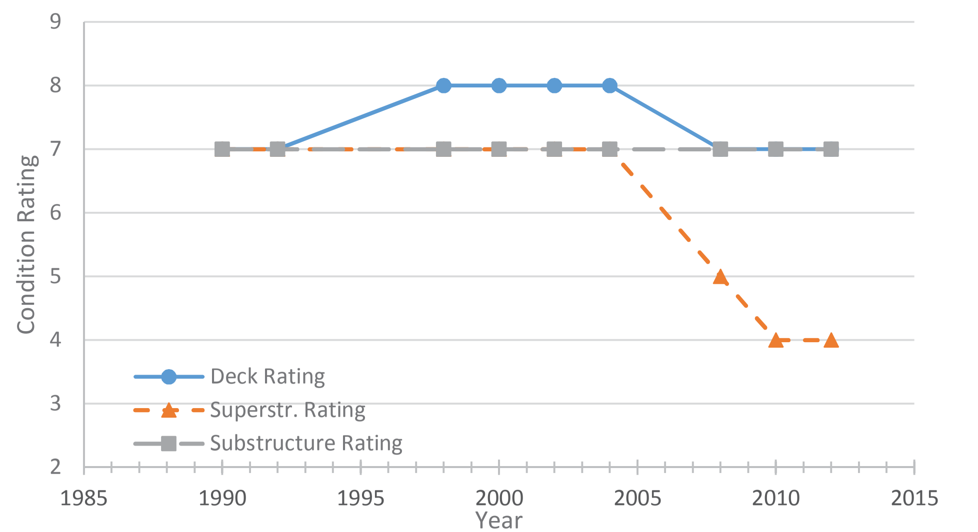
Bridge ID: I65-140-4117 DNBL	
Year Built	1958
No. of Spans	3
Bridge Length (ft.)	204.1
Max. Span Length (ft.)	68.0
Superstructure Type	CRCG
ADT	25400
ADTT	31%
No. of Lanes	2
Location	Boone County
Environment	Central District
Deck CR (2013)	5
Superstructure CR (2013)	4
Substructure CR (2013)	4
Facility Under	Highway
Clearance	15’-10”
Scour Rating	N/A
Skew (Degrees)	55
Controlling Damage Mode	Corrosion



Year	1993	1995	1996	1998	2000	2003	2005	2007	2009
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months
Year	2011	2012	2013						
Inspection Interval	24 months	24 months	24 months						

Bridge ID: I70-77-05646 B

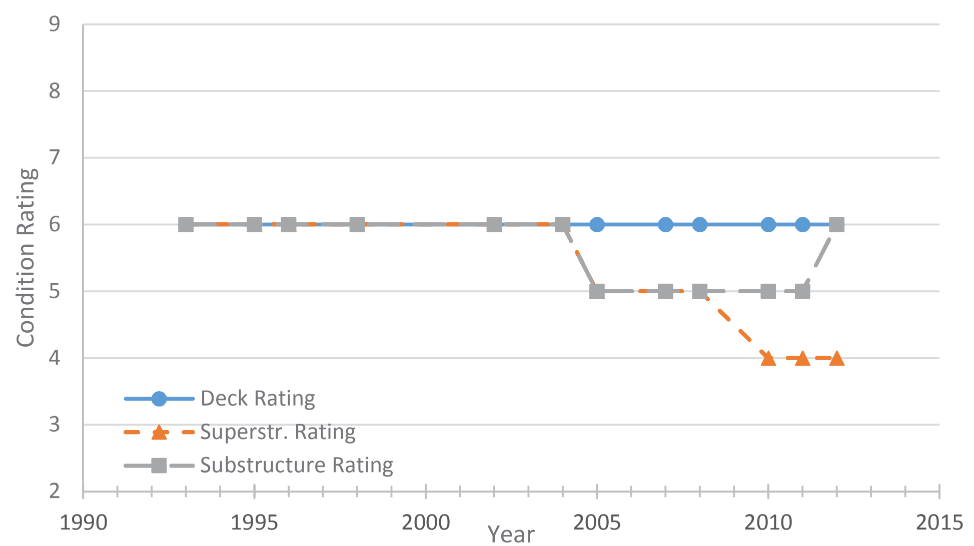
Year Built	1973
No. of Spans	3
Bridge Length (ft.)	177.8
Max. Span Length (ft.)	82.0
Superstucture Type	CPCIB
ADT	138000
ADTT	5%
No. of Lanes	7
Location	Marion County
Environment	Central District
Deck CR (2013)	7
Superstructure CR (2013)	4
Substructure CR (2013)	7
Facility Under	Highway
Clearance	21'-08"
Scour Rating	N/A
Skew (Degrees)	2
Controlling Damage Mode	Corrosion



Year	1990	1992	1998	2000	2002	2004	2008	2010	2012
Inspection Interval	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months	24 months

Bridge ID: I74-30-4942 CEBL

Year Built	1964
No. of Spans	3
Bridge Length (ft.)	125.0
Max. Span Length (ft.)	38.0
Superstructure Type	RCG
ADT	8000
ADTT	43%
No. of Lanes	2
Location	Montgomery County
Environment	Central District
Deck CR (2013)	6
Superstructure CR (2013)	4
Substructure CR (2013)	6
Facility Under	Waterway
Clearance	N/A
Scour Rating	Fixed
Skew	32
Controlling Damage Mode	Corrosion



Year	1993	1995	1996	1998	2002	2004	2005	2007	2008
Inspection Interval	48 months	48 months	48 months	48 months	48 months	24 months	24 months	24 months	24 months
Year	2010	2011	2012						
Inspection Interval	24 months	24 months	24 months